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(ACI PROCEEDINGS Vol. 51)

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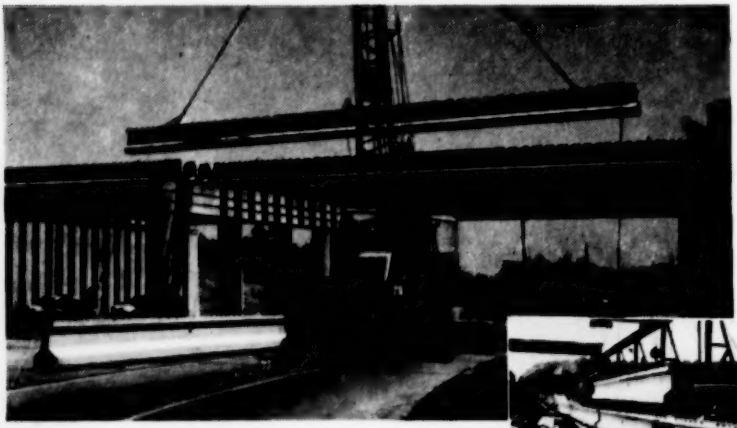

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Vol. 26—No. 2 October 1954 (Proceedings Vol. 51)

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Contributions to the Journal

The ACI JOURNAL technical pages are a medium for reporting what is of current interest in the concrete field; in research, design, construction, maintenance, or manufacturing. Members and readers are urged to "have their say" as a means of sharing knowledge and experience; time and changing interests demand that new names and new ideas take their places beside the old. Contributions describing new materials and techniques or new applications of familiar practices are welcomed by the Institute.

To guide the flow of material the Board of Direction has assigned to the Technical Activities Committee the selection of papers, committee reports, discussions, and other contributions for JOURNAL publication. To assist in their technical appraisal a corps of experts in various segments of the field volunteer their services. From the technical standpoint, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

General considerations by TAC are whether the contribution will be of interest and service, will fit into a well-balanced publications schedule, and can be published at reasonable expense to the Institute.

To expedite appraisal and publication, if accepted, contributors are requested to furnish glossy prints or inked tracings of illustrations accompanying manuscripts submitted for consideration. Further details of form and physical arrangement of contributions begin on p. 14, 1953 *ACI Directory*. Separate prints of the "publications policy" are available free upon request to the Secretary.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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In ACI Journal next month

"Model Analysis of a Skewed Rigid Frame Bridge and Slab," by D. H. PLETTA and D. FREDERICK, presents results of an experimental analysis of a small aluminum model of a skewed-slab rigid-frame bridge. A model theory is also checked using reinforced concrete as the model material by testing three sizes of geometrically similar simple beams and a model of a skewed slab with curbs. The slab is a $\frac{1}{8}$-scale model similar to a $\frac{1}{2}$-scale model tested previously at the University of Illinois. Results indicate that small reinforced concrete models do behave just as their prototypes up to ultimate load, and it now seems possible to test models of entire structures rather than just their component parts as has been the practice in the past.

GEORGE L. KALOUSEK presents a brief resumé of some studies on the problem of shrinkage and attendant cracking of concrete masonry in his paper "Fundamental Factors in the Drying Shrinkage of Concrete Block." New data are presented and interpretations of different observations on the drying shrinkage phenomenom are advanced.

GORDON W. GREENE discusses the Schmidt concrete test hammer in his paper "Test Hammer Provides New Method of Evaluating Hardened Concrete." The test hammer provides a convenient method of determining the strength of nearly any concrete mass that has a smooth, flat surface. The test is nondestructive. In this paper results of tests made with the instrument are compared with results of compression and flexure tests.

In "Ultimate Load Theory and Tests of Cylindrical Long Shell Roofs," by G. C. ERNST, R. R. MARLETTE, and G. V. BERG, an ultimate load theory for thin shell roofs, developed independently by A. L. L. Baker and K. W. Johansen, is presented with modifications as applicable to the design of long, simply-supported, cylindrical shells. The basis of analysis is similar to a rupture theory by H. Lundgren. In addition to design equations for long cylindrical shells, results from exploratory tests are presented in confirmation of the theory. The theory and equations are intended to supplement, not supplant, existing elastic theory.

"Accidental Air in Concrete," by M. F. MACNAUGHTON and JOHN B. HERBICH deals with instances where accidental air in concrete was found to be present in amounts far beyond those normally encountered in plain mixtures. In connection with one specific instance they outline the investigation of the problem and the measures adopted to control vagrant and final air content in the concrete mixtures on the work within reasonable and normally acceptable limits.

Apparatus and methods for measuring the permeability of portland cement pastes are described in "Permeability of Portland Cement Paste," by T. C. POWERS, L. E. COPELAND, J. C. HAYES, and H. M. MANN. Test results are given showing the effects of curing, cement content, cement composition, and cement fineness. Also, data on some natural stones are compared with data on hardened pastes.

Admixtures for Concrete*

Reported by ACI Committee 212

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SYNOPSIS

With the aim of providing a perspective of the field of admixtures for the engineer confronted with a need of modifying concrete to meet special requirements of a given job, admixtures are classified into 11 groups. The 11 groups are: (1) accelerators, (2) retarders, (3) air-entraining agents, (4) gas-forming agents, (5) cementitious materials, (6) pozzolans, (7) alkali-aggregate expansion inhibitors, (8) dampproofing and permeability reducing agents, (9) workability agents, (10) grouting agents, and (11) miscellaneous. Discussions are given of the factors which might indicate the usefulness of admixtures of each group, and of the important effects which may ordinarily be expected from the use of materials of each group.

INTRODUCTION

This report is an expansion and continuation of Committee 212's November, 1944, report[1] and aims to summarize present knowledge of the advantages and limitations of admixtures. It is recognized that the classification of admixtures is difficult due to the fact that there are many varieties and types of admixtures and also that some admixtures produce more than one effect. The committee has decided that the classification used in its original 1944 report is the most logical one and it is followed with few minor changes in the present report.

The committee endorses the statement of P. H. Bates[2] that "the use of admixtures should not be frowned upon or scorned. On the contrary, their development and study should be highly commended and fostered." From a historical standpoint certain materials now used as admixtures preceded the use of portland cement by several thousand years. On the other hand, other admixtures are of recent usage and are appearing on the market in ever-increasing number as in the case of certain surface active agents used for the purpose of entraining air or accomplishing other effects.

*Received by the Institute Feb. 15, 1954. Title No. 51-5 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 2, Oct. 1954, *Proceedings* V. 51. Separate prints in covers are available at 75 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1955. Address 18263 W. McNichols Rd., Detroit 19, Mich.

This report in form and substance as here submitted was approved unanimously by the committee as listed above.

An admixture is defined in ASTM Designation C 125 as: "A material other than water, aggregates, and portland cement (including air-entraining portland cement and portland blast-furnace slag cement) that is used as an ingredient of concrete and is added to the batch immediately before or during its mixing."

Usually an admixture is used to modify the properties of the concrete in such a way as to make it more suitable for the work at hand. Under certain conditions, use of a suitable admixture may impart desirable characteristics which cannot be secured as economically by other methods.

Some of the more important modifications of properties of concrete which have been the objective of the use of admixtures are:

(a) Improvement of workability
(b) Acceleration of the rate of strength development at early ages
(c) Retardation of initial stiffening or too rapid set
(d) Retardation or reduction of heat evolution
(e) Reduction in bleeding
(f) Increase in durability or in the resistance to special conditions of exposure
(g) Control of alkali-aggregate expansion
(h) Decrease in capillary flow of water
(i) Decrease in the permeability to liquids
(j) Production of cellular concrete
(k) In grout mixtures, the improvement of penetration and pumpability and the reduction of segregation
(l) Prevention of settlement or creation of slight expansion in concrete and mortar used for filling blockouts or other openings in concrete structures, and in grout for seating machinery, columns, and girders
(m) Increase in bond to steel reinforcement

In considering the effects of admixtures in concrete it is pointed out that: (a) at times a change in type of cement or amount of cement used, or a modification of aggregate grading or mix proportions may offer the surest and most economical approach to the objectives desired; (b) many admixtures affect more than one property of concrete, sometimes affecting desirable properties adversely; (c) the effects of some admixtures are significantly modified by such factors as wetness and richness of mix, by aggregate grading, and by character and length of mixing; (d) the specific effects of some admixtures vary with the type and with the brand of cement used; and (e) accordingly, specific effects which will result from the use of an admixture can seldom be predicted accurately.

ECONOMIC ASPECTS OF THE USE OF ADMIXTURES

Use of an admixture may increase the cost of the concrete. Therefore, even though a given admixture may produce a desirable effect, the value of that effect should be weighed against its cost. Moreover, the effect of a given admixture can often be obtained, at least in some degree, by other

means or by other admixtures. Hence, whenever possible, the cost of an admixture should be compared with that of alternative materials or methods for getting the desired result. Conversely, the economic gains possible through decreased costs by the use of admixtures must also be considered.

In evaluating an admixture, its effect on the volume of a given batch should be noted. If adding the admixture changes the volume, as is often the case, the change in the properties of the concrete will be due not only to direct effects of the admixture but also to the changes in the amounts (per cubic yard of concrete) of the original ingredients. If the admixture increases the volume of the batch, the admixture must be regarded as effecting a displacement either of part of the original mixture or of the original ingredients—cement, aggregate, or water. All such changes in the composition of a unit volume of concrete must be taken into account when testing the direct effect of the admixture itself, and in estimating the cost of the admixture.

Other items entering into the evaluation are the cost of handling an extra ingredient, and any effect the use of the admixture may have on the cost of transporting, placing, and finishing the concrete.

Usually the evaluation of any given material should be based on the results obtained with the particular concrete in question. This is highly desirable since the results obtained are influenced to an important degree by the characteristics of the cement and aggregate and their relative proportions.

CLASSIFICATION OF ADMIXTURES

A division of the admixture field into 11 groups, according to type of materials constituting the admixtures, or to the characteristic effects of their use, is presented along with brief statements of the general purposes and expected effects of the use of materials of each group. The wide scope of the admixture field, the continuous entrance of new or modified materials into this field, and the variations of specific effects with different concreting materials and conditions, precludes a detailed listing of commercial admixtures and their effects on concrete. The groups are as follows:

(1) Accelerators
(2) Retarders
(3) Air-entraining agents
(4) Gas-forming agents
(5) Cementitious materials
(6) Pozzolans
(7) Alkali-aggregate expansion inhibitors
(8) Dampproofing and permeability reducing agents
(9) Workability agents
(10) Grouting agents
(11) Miscellaneous

Commercial admixtures may contain materials of two or more of the above group. For example, a workability agent which has a retarding action may

be combined with an accelerator to compensate for the retarding action. Those types of admixtures possessing properties identifiable with more than one class are considered in the discussion following as belonging to the class that describes the most prominent or important effect on the concrete.

ACCELERATORS

General

Accelerators may be added to concrete to increase the rate of early strength development in concrete to: (a) permit earlier removal of forms; (b) in certain types of work, reduce the required period for curing; (c) advance the time when a structure can be placed in service; (d) with proper protection, partially compensate for the retarding effect of low temperatures during cold weather concreting; and (e) reduce the period of protection required for initial and final set in emergency repair and other work. In many cases, the engineer must decide between (a) using an admixture; (b) increasing the cement content; (c) using high early strength cement; (d) providing greater protection or longer curing; or (e) any combination of these alternatives.

Chemical materials which accelerate the normal reactions between portland cement and water include calcium chloride[3,4] (the most commonly used accelerator), some of the soluble carbonates, silicates and fluosilicates, aluminous cements, and some organic compounds such as triethanolamine.

Accelerators such as the fluosilicates and triethanolamine are comparatively expensive. The accelerating effect of triethanolamine can be pronounced—0.2 percent added to some cements can produce a flash set. General recommendations on the use of accelerators other than calcium chloride cannot be given because present information about other types is inadequate. In general, other types should be used only on the basis of competent technical advice, adequate preliminary tests, or satisfactory experience with the material in question. With some accelerators, small changes in the amount added can cause large differences in their effect. Some are capable of reducing the period during which the concrete remains plastic to less than 10 min. Such acceleration may lower the ultimate strength markedly, however, and is used only for emergency patching or stopping of leaks.

Aluminous cements are greatly accelerated by the addition of small amounts of portland cement. In this case portland cement is, for all intent and purpose, being used as an admixture with aluminous cement. Since the accelerating effect is partly due to the lime in the portland cement, the amount of portland cement required to produce a certain time of set is dependent upon the particular cement used. The combination normally recommended for field use is about 70 percent aluminous cement and 30 percent portland cement. With particular cements, 60 percent aluminous cement may produce a flash set in less than 10 min. while 70 percent aluminous cement may produce a final set between 2 and 3 hr. It has also been claimed that the

setting time of portland cement may be accelerated, through varying degrees, by the use of minor amounts of aluminous cement (5 to 20 percent).[5]

Calcium chloride

Commercial chloride[3,4] (commercial flake calcium chloride which contains 77–80 percent $CaCl_2$ or pellet calcium chloride containing 90–95 percent $CaCl_2$) generally can be used safely in amounts up to 2 and not more than 3 percent by weight of the portland cement. Laboratory tests have indicated that the increase in strength resulting from the use of 2 percent of calcium chloride may range from 400 to 1000 psi at 1 through 7 days for 70 F curing. Numerous tests at 40 F indicate that, although the strengths developed through 7 days are lower, the increase in strengths obtained with 2 percent calcium chloride are of the same order as those obtained at 70 F. The increase in strength usually reaches its maximum in 1 to 3 days and thereafter generally decreases. At 1 year, some increase still continues in concretes made with most cements. The specific effect of the use of calcium chloride varies, however, for different brands of cement, as indicated by the range of strength increases cited above for the early ages.

Minor effects resulting from the use of calcium chloride include a small increase in the workability of the fresh concrete, an early commencement of the initial stiffening with some cements, and accordingly, a reduction in bleeding and probable increase in the drying shrinkage of the hardened concrete. Data are conflicting on the effect of calcium chloride on drying shrinkage. However, the shrinkage is generally increased when calcium chloride is used. Differences obtained by various investigators may be due to differences in curing procedure. The rate of heat evolution is increased materially at early ages, and consequently, where temperature differentials within the concrete are important factors, the effect of this property of accelerators should be taken into consideration before they are used. The total heat liberated is not appreciably changed. Also, the use of accelerators in warm concretes, such as may be obtained in hot weather concreting, may result in such rapid stiffening as to impede placement or finishing.

Calcium chloride may be especially beneficial for concrete exposed to low or freezing temperatures at early ages. It increases the rate of early heat development, accelerates the set, but lowers the freezing point of the concrete only slightly. It generally increases expansion under moist curing; it generally increases shrinkage but decreases moisture loss under drying conditions. It increases the expansion caused by the alkali-aggregate reaction but when the latter is effectively controlled by the use of both low-alkali cement and pozzolan, the effect of calcium chloride appears to be unimportant. It also lowers the resistance of concrete to sulfate attack. Freezing and thawing durability is increased at early ages by calcium chloride, but is somewhat reduced at later ages. Calcium chloride does not cause corrosion of embedded steel. It significantly increases the resistance of concrete to erosive and abrasive action.

ACI Committee 617, in the ACI Standard "Specifications for Concrete

Pavements and Bases (ACI 617-51)," states: "Calcium chloride may be added either dry or in solution. When used in solution, it is convenient to so proportion the solution that 1 quart contains 1 lb of calcium chloride." The use of calcium chloride in solution is preferable where excellent inspection control is maintained. Where the solution is used, care must be taken to insure proper mixing. Calcium chloride should be added to water and not water to calcium chloride since a coating may form which is difficult to dissolve. Checks should be made on the specific gravity of the solutions to be certain that proper strength of solution is being maintained.

Where calcium chloride is to be used dry, it may be measured by volume or by weight. In using the dry material, care must be taken to insure that the material is not caked due to poor storage conditions. Storage conditions required for portland cement are adequate for the storage of calcium chloride. The use of calcium chloride in the dry form is not advised. However, it has been successfully employed by screening the material through a ¼-in. sieve prior to use to preclude lumps since undissolved lumps may result in excessive concentrations of calcium chloroaluminate with resultant pop-outs. When placed in the skip, calcium chloride should not be placed directly in contact with cement, but put in with the aggregate.

Use of accelerators in concrete products

Calcium chloride, as well as other materials which accelerate hardening and promote early strength development of concrete, may prove advantageous in the manufacture of a variety of concrete products. Early attainment of strength in a building block, for example, reduces the curing period, compensates in part for slow hardening in cold weather, and decreases the time required to produce a fully natured block. Similar advantages may be obtained in the manufacture of other concrete products. During warm weather, accelerators should be used judiciously so as not to produce too rapid a set. Many plants employ only high temperature curing during summer months, and a combination of high temperature curing with an integral accelerator during the winter months.

RETARDERS

General

The principal uses in concrete of admixtures having a retarding effect[6] on the set of cement are to overcome the accelerating effect of temperature during hot weather concreting operations, to remove the tendency of some cements to exhibit false set, and to delay the early stiffening action of concrete for difficult conditions of placing. In addition, solutions are sometimes applied to forms to inhibit the set of a surface layer of mortar so that it can be readily removed by brushing, thus exposing the aggregate and producing unusual surface texture effects. There are several commercial formulations on the market for this purpose.

Retarders are especially useful in cement grout slurries, particularly where it may be necessary to redrill grout holes, for grouting over a prolonged

period of time, in cases where the grout must be pumped for a considerable distance, and where hot water flows are encountered. These materials are organic and include the commercial products RDA, Ray lig binder, and SRDA. Slurries used in cementing oil wells must remain fluid for periods of one or more hours at temperatures up to 400 F and pressures up to 18,000 psi. Often this condition is met through the use of admixtures which will retard the setting of the cement. A considerable number of materials have been found useful for this purpose and usually fall within the following classes: (1) starches or cellulose products, (2) sugars, and (3) acids or salts of acids containing one or more hydroxyl group.

A wide variety of chemicals are mentioned in current literature as having a retarding influence on the normal setting time of portland cement. Some of these have been found variable in action, retarding the set of certain cements and accelerating the set of others. Some chemicals act as retarders or inhibitors when used in certain quantities, and accelerators when used in other amounts. The effects of most of these materials on the other properties of concrete and mortar are not well known, although, in general, some reduction in early strength accompanies the use of organic retarders.

Unless experience has been had with a retarder, its use as an admixture should not be attempted without technical advice or preferably advance experiment with the cement and other concreting materials involved, to determine the extent of its effects on the setting time and other properties of the concrete. The more commonly known retarders used as admixtures, such as carbohydrate derivatives and calcium lignosulfonate, are employed in only small fractions of a percent (0.2 or less) by weight of the cement.

Application of retarders in false set

In view of the importance of minimizing or controlling false set in cement, it is desirable to discuss the use of admixtures for this purpose in some detail. It is generally agreed that any appreciable, sudden, or abnormal stiffening far in advance of the time of the initial set, is a false set. This phenomenon, which occurs sporadically, has been known by several other names, such as gum set, plaster set, rubber set, hesitation set, and premature stiffening. It can frequently be overcome by continued mixing or by reworking the paste, mortar, or concrete after completion of mixing. Early methods for the quantitative evaluation of false set employed the slump cone, the test being made at frequent intervals following completion of mixing of the concrete. At the present time this property is usually measured on cement pastes by using the Vicat apparatus.[7]

The causes of false set[8] are generally related to the process of manufacture of the cement or conditions to which the cement may be subjected prior to delivery, and for this reason the correction of false set should be carried out at the point of inception of the cause. It is generally agreed that the basic cause of false set is the reduction in the effective content of gypsum or of sulfuric anhydride in proper form, available to act as retarder. A portion

of the gypsum initially added may have been dehydrated to a condition rendering that portion ineffective in retarding.

When false set difficulties are being experienced and it is necessary, due to job conditions, to take remedial steps, certain admixtures may be used. Probably the simplest and cheapest corrective measure which can be used on the job is the addition of an appropriate amount of powdered gypsum, generally 1 to 2 percent by weight of cement. The appropriate amount of gypsum must be determined on the job and, as previously stated, will vary with materials and job conditions. Use of gypsum for the purpose of reducing false set is only recommended where adequate inspection and control is available because the addition of an excessive amount may cause undesirable expansion in the concrete. In addition, wetting agents or combined wetting and air-entraining agents are available which, when used in small quantities, are sometimes effective in overcoming the false set observed for the cement without admixture. Other admixtures which have been successfully used (in very small quantities) are mucic acid, calcium acetate, and the commercial products Ray lig binder, SRDA, and spray dried RDA. However, the appropriate admixture, both with respect to type and quantity, will be a matter of test for the particular application.

Although indiscriminate use of admixtures to correct false set is not recommended, the committee recognized the applicability of admixtures where economy dictates that remedial measures be employed at the job site. Such utilization, however, will require preliminary testing employing the same materials and mixes as are to be used on the job, and control tests at frequent intervals will be required to insure that the corrective measures remain adequate. It is the consensus of the committee that, in the interest of economy, false set should be corrected before shipment of the cement to the job.

AIR-ENTRAINING AGENTS

Effects of air entrainment

During the past 15 years the attitude of the engineering profession toward admixtures in general and admixtures for improving durability in particular has undergone a marked change. This has been due, to a large extent, to the discovery of the benefits of air entrainment.[9,10,11] It was formerly believed that the best protection against damage by frost action was to use a dense, impermeable mix well placed and thoroughly cured. These basic principles still hold. However, it is now known that even when all of these rules are followed, resistance to frost action can be still further improved by incorporating into the concrete a suitable amount of entrained air. Furthermore, it is becoming increasingly evident that this additional protection is needed especially in cases where freezing conditions are severe or where (as in the case of pavements) common salt or calcium chloride is used for ice removal. Experience in both laboratory and field during the last 15 years has demonstrated the superior durability of air-entrained concrete and there should be no hesitancy in using it under conditions where severe natural weathering is a problem.

Air entrainment materially alters the properties of both the plastic and the hardened concrete. Air-entrained concrete is considerably more plastic and workable than ordinary concrete. It can be handled and placed with less segregation and there is less tendency for bleeding and water gain. These properties indirectly aid in promoting durability by increasing uniformity and eliminating planes of weakness at the top of vertical lifts. However, the major effect is due to a change in the characteristics of the concrete brought about by the presence of air bubbles in the paste. At a given air content, the protectiveness of the voids is greater the greater the number of voids per unit volume of paste. This means that the voids are more effective the closer they are together. The mechanism of their effect is too complex to be discussed here.[12]

Air entrainment, while improving both workability and durability, may have an adverse effect upon strength. Within the range of air contents normally used, the decrease in strength usually is about proportional to the amount of air entrained. However, for most types of exposed concrete a slight reduction in strength is more than balanced by the improved resistance to frost action. Moreover, the reduction in strength will rarely exceed 15 percent in the case of compressive strength and 10 percent in the case of flexural strength. These figures are for equal cement content and with the sand and water content of the air-entrained concrete reduced to the extent permitted by the increased workability of this type of mix.

The discussion above refers to the use of moderate amounts of entrained air, usually 7 percent by volume or less. In many applications of concrete, particularly in precast units, much greater quantities of entrained air are employed to produce lightweight products with superior thermal insulating properties. This application will be mentioned later.

Air-entraining admixtures

There are now on the market a large number of commercial admixtures for which air-entraining properties are claimed and some of these are being used extensively. Many materials, including the natural wood resins, fats, various sulfonated compounds, and oils, have the property of entraining air in concrete. Some of these materials, as for example the resins, are insoluble in water and must be "neutralized" before they can be used as admixtures. Soluble salts of the resins are now available commercially so that it is no longer necessary to neutralize them in the field.

Before specifying or permitting the use of an air-entraining admixture in concrete, the user should satisfy himself (1) that the material actually does function as an air-entraining agent and (2) that none of the essential properties of the concrete (*e.g.*, strength, volume change) are seriously impaired. The American Society for Testing Materials has developed a tentative specification and a method of testing air-entraining admixtures for concrete (ASTM Designations C260 and C233).

Air-entraining cement

Air-entrained concrete can also be made by using an air-entraining port-

land cement. Air-entraining portland cement is portland cement that contains air-entraining agent which has been interground with it during its manufacture. The American Society for Testing Materials has issued a specification (C175) for this type of cement.

Choice of method

At present both methods of entraining air are being used extensively and as judged by experience records extending over 15 years both are giving improved concrete. Some engineers prefer adding the agent at the mixer because the amount of agent can be readily changed. Others prefer air-entraining cement because of the convenience of using it.

Moderate air entrainment in concrete products

Quantitative data in the literature are lacking, but the improved workability resulting from entrained air has been credited with several advantages in the manufacture of concrete block. With any of the usual manufacturing methods, the use of air-entrained concrete permits greater compaction resulting in denser block. Appearance of the block is improved, edge tear is reduced, and the block strip cleanly with sharp edges and corners. Of equal importance is the improvement in surface texture which may be obtained especially where aggregates deficient in fines have been employed. Block containing entrained air also have less absorption and present greater resistance to water passage.

Of prime concern to the block manufacturer is a reduction in breakage of green block. In block manufacture, a dry mix must be employed since the product is removed from the mold immediately after compaction. Addition of an air-entraining agent permits the use of more water, thus giving a water-cement ratio closer to the optimum value for maximum strength with the given proportions of dry materials. The fresh block have a "rubbery" consistency which results in fewer cracks and less breakage during handling at early ages.

Similarly, satisfactory results are obtained with other concrete products. In cast stone the plasticizing effect of entrained air reduces bleeding and segregation and makes possible more exact reproduction of mold contours.

Use of air-entrained concrete in fabrication of pipe results in better flow of concrete around reinforcing, less tendency to displace reinforcing during compacting processes, easier stripping of molds without distortion of the pipe, sharper edges, better appearance, and less permeability.

In some installations of concrete products, such as cribbing and curbing, there is considerable exposure to freezing and thawing action. As already pointed out the use of adequately prepared and controlled air-entrained concrete is the best guarantee of resistance to this destructive action.

Use of lightweight aggregates in concrete products is increasing rapidly. Lightweight aggregate mixtures are generally harsh, hence use of air entrainment with them is particularly advantageous. In many cases the reduction in bleeding and cement segregation resulting from air entrainment is great.

In general, beneficial results may be obtained by the addition of entrained air in both lean and rich mixes with any grading or type of aggregate including those with light weight. The greatest improvement is obtained, however, in harsh mixtures deficient in fines.

The simplest method of obtaining entrained air is by the use of an air-entraining cement. Such a cement might, where necessary, be blended with a non-air-entraining cement to get closer control in the final mixture. For best results it is probably desirable to add an air-entraining admixture at the concrete mixer, because different amounts are necessary to produce the optimum results in various products and methods of making the same product. Relatively large amounts of air-entraining admixtures are needed for best results in making block while small amounts are preferred in cast stone manufacture. The packer-head process of pipe manufacture benefits from relatively large amounts of air entrainment, the dry-tamp process from small amounts. The proper amount in any particular case must be determined by experiment. Care should be exercised to avoid excessive air entrainment since the strength of the product may be greatly reduced thereby.

Lightweight concrete by air entrainment

The strength of lightweight concrete decreases generally with decreased unit weight while the insulation value increases. In the application of lightweight concretes, it is necessary to weigh the advantages of high insulating value and reduced weight against the disadvantage of low strength. Load bearing walls require higher strengths than those which bear no loads and in certain insulating applications strength is a minor factor.

Unit weights may be reduced by using hollow building units and by the substitution of lightweight for conventional aggregates. Lightweight aggregates have good insulating properties because they contain small air voids. An alternate method involves the substitution of small air or gas bubbles for all or part of the aggregate.[13] Concretes of this type are known as gas, foam, or cellular concretes.[14] Such materials are used extensively in Europe and the interest in them in this country is increasing.

Cellular concretes may be divided into the general groups gas concrete and foam concrete; both are produced through the use of admixtures. Gas concrete will be discussed in the next section.

Air may be entrained in amounts from 30–60 percent by volume for structural concrete and 70–85 percent for insulation. The air may be whipped into the mass with rapid agitation and the addition of air-entraining agents such as sodium lauryl sulfate, alkyl aryl sulfonate, certain soaps, resins, and other agents. The quantities used must, of course, be much greater than those employed in the usual air-entrained concrete.

In another process the foam is produced separately using agents of the types employed to combat gasoline fires such as hydrolyzed waste protein. In such an operation the foam is added to the mixer in place of the conventional aggregate.

The various foam concretes may vary considerably in properties, ranging from materials weighing as little as 20 lb per cu ft and containing no mineral aggregate, to those weighing 110 lb per cu ft or more and containing conventional aggregate. The lightest materials may possess just sufficient strength to retain their shape in handling and are used primarily for acoustic or thermal insulation purposes. Some of the heavier ones may have sufficient strength for structural applications as in walls of dwellings. Dilnot[15] has outlined the advantage of foam concrete as compared to gas concrete from the standpoint of control problems and cost.

When cellular concretes are employed in the manufacture of concrete products, the advantages of autoclaving are considerable. A comparison of 40 lb per cu ft block made with normal-curing and with similar-weight autoclaved block shows the latter having slightly better insulating properties, three times the strength, one-quarter the drying shrinkage, and one-half the wetting movement.[15]

This reduction of drying shrinkage and wetting movement is quite important. The dimensional stability of ordinary concretes which are subjected to moisture changes is provided by the aggregates, which resist volume changes. When conventional aggregates are replaced by foam this stabilizing factor is lost and hence any process which will reduce the wetting movement is desirable.

Autoclaved products with unit weights of 35–50 lb per cu ft have been reported which had strengths of from 500 to 850 psi, while those of 70–90 lb per cu ft had strengths up to 2000 psi. Coefficients of thermal conductivity ranged from 0.75 Btu per in. per sq ft per hr per deg F for 30 lb per cu ft concrete to 2.3 for that of 90 lb per cu ft.

The use of autoclaving suggests the addition of a pozzolanic material to the mix. Experimental concrete has been prepared using one part of cement to two parts of fly ash by absolute volume and with sufficient foam to produce 50 percent voids. This is a concrete which might be considered principally admixture.

Attention may also be directed to the so-called "no-fines" concrete in which 20 to 30 percent entrained air replaces the sand in a mix using pea gravel as the coarse aggregate.[16] This material, suitable for insulating fills and dwelling walls, has unit weights ranging from 105 to 118 lb per cu ft and compressive strengths from 200 to 1000 psi.

GAS-FORMING AGENTS

Use of gas-forming agents to counteract subsidence and bleeding

Settlement shrinkage and bleeding in plastic concrete are caused by gravitational settling of the individual solid particles in a semiliquid mixture. The extent of settlement or bleeding is dependent on several factors and, when excessive or under certain concreting conditions, may result in undesirable characteristics in hardened concrete or mortar. Accumulation of

low quality matrix films, laitance layers, and voids on the underneath side of forms, blockout cavities, reinforcing steel, or other embedded parts, or under machinery bases may reduce bond, watertightness, uniformity, and strength in the concrete or mortar and necessitate, in some instances, costly clean-up and grouting operations.

Aluminum powder,[17,18] when added to mortar or concrete, reacts with the hydroxides present in hydrating cement to produce minute bubbles of hydrogen gas throughout the cement-water matrix. The amount and extent of the aluminum reaction is dependent upon the type and amount of aluminum powder, fineness and chemical composition of the cement, temperature, mix proportions, and other factors. Usually, the unpolished powder is preferred, though when a slower reaction is desired, the polished form may be advantageous. The amounts added are usually limited to 0.005 to 0.02 percent by weight, of cement, though larger amounts may be used in the production of lightweight concrete of low strength.

The action of aluminum powder, when properly controlled, causes a slight expansion in plastic concrete or mortar and thus reduces or eliminates settlement and may, accordingly, increase bond to horizontal reinforcing steel and improve the effectiveness of grout in filling joints. It is particularly useful for grouting under machine bases or backfilling under horizontal surfaces where complete filling of the cavity is essential. The effect on strength depends to a considerable extent on the degree to which the tendency of the concrete to expand is restrained. Without restraint the loss of strength may be considerable, but with complete restraint imposed, the strength is not affected appreciably and, in some cases, may be slightly increased. It is, therefore, important that the forms be tight and that they completely confine the grout or concrete treated with aluminum powder. Ordinarily, the density is slightly reduced and the resistance of the hardened concrete to frost action is improved.

In hot weather, the action of aluminum powder may occur too quickly and its beneficial action be lost. In cold weather, the action is much slower and may not progress fast enough to produce the desired effect before the concrete has set. The delayed generation of gas may or may not be harmful depending on the rate and amount of gas generation after the concrete has set and upon the degree of restraint imposed. At normal temperatures, the aluminum reaction starts at the time of mixing and may continue for 1½ to 4 hr. At temperatures above 90 F, the reaction may be completed in 30 min. and subsidence may take place until the concrete takes its initial set. At 40 F, the reaction may not be effective for several hours. Approximately twice as much aluminum is required at 40 F as at 70 F to produce the same amount of expansion.

As very small quantities of aluminum powder are generally used (about 1 teaspoonful per sack of cement), and as it has a tendency to float on the mixing water, the powder is generally pre-mixed with fine sand or pozzolan.

Use of gas-forming agents in concrete products

By using larger quantities of aluminum powder than indicated above, lightweight concretes can be produced. In some cases additional alkali such as sodium hydroxide or trisodium phosphate is added to accelerate the reaction. If the latter is controlled so that the gas is formed at the proper time and in the desired amounts, the mixture increases greatly in volume. It has also been found desirable to add some type of emulsifying agent to stabilize the mix, thus reducing the tendency toward gas bubble coalescence with resultant segregation and formation of layers. By varying the proportions and carefully controlling other factors, such as temperature and the uniformity of the cement used, it is possible to produce concretes over a wide range of density. Such products are made either with or without aggregates. The process is suitable not only for concrete products, but also in some in situ work such as insulation fills where possible cracking is unimportant.

Aluminum powder is used for this purpose in amounts of approximately ¼ lb per bag of cement, but it is usually necessary to make trial runs when any of the ingredients or manufacturing conditions change. Zinc and magnesium powders are also used for this purpose and a chemical combination which produces oxygen instead of hydrogen bubbles in the mass consists of hydrogen peroxide and bleaching powder. Patents have been issued for processes which produce chlorine and nitrogen.

In general, the properties of the resulting gas concrete are similar to those of comparable unit weight produced with foam or high air entrainment. For information on properties, reference should be made to the preceding section where the latter products were discussed.

CEMENTITIOUS MATERIALS

This group includes natural cements, hydraulic limes, and slag cements (mixtures of blast-furnace slag and lime). When used, materials of this class are usually substituted for 10 to 25 percent, by weight, of the portland cement, although substitution of larger amounts are made when lowered strengths are not objectionable. Effects resulting from their substitution for part of the cement[19] generally are an increase in workability (more pronounced for harsh mixes) and decreases in bleeding, in segregation, in heat of hydration, and for most of the materials, a decrease in strength. Natural cementing materials are generally considered as not having significant effect on frost resistance unless an air-entraining agent also is present in the cement.

The use of materials of this class may require an increase in mixing water and, accordingly, an increase in drying shrinkage may result. A relatively long curing period is needed for development of potential strength.

POZZOLANIC MATERIALS

A pozzolan is defined in ASTM Designation C129 as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemi-

cally react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." These materials are sometimes used in large hydraulic structures where it is desirable to avoid high temperatures or in structures exposed to sea water or sulfate-bearing waters.[29] When used, pozzolanic materials generally are substituted for 10 to 35 percent of the cement. They generally have a lower specific gravity than portland cement; therefore, if substitution is made on a weight basis a greater bulk volume of cementitious material results. Substitution of pozzolanic material for part of the cement generally reduces permeability to water and increases resistance to aggressive attack of sea water, sulfate-bearing soil solutions, and natural acid waters. These effects are greater for lean mixes than for rich mixes.

The effect of pozzolan upon the strength of portland-pozzolan concrete varies markedly with the particular pozzolan used. Generally the strength development is relatively slow, even more so at low temperatures. Under favorable curing conditions, later strengths, as a rule, will be higher than with portland cement alone although large differences result from the use of different pozzolans. In all cases, prolonged wet curing is necessary for development of potential strength.

The addition of pozzolanic materials to concrete (instead of substitution for part of the cement) improves workability, impermeability, and resistance to chemical attack in a manner similar to that resulting from substitution. These improvements are most marked when the pozzolan is added to a concrete originally deficient in the amount of fine materials; if added to one already containing an abundance of fine material, the water-cement ratio must be increased, and accordingly absorption and drying shrinkage may be increased and resistance to freezing and thawing may be reduced.

The effects of additions on strength vary with the mix and the material. Generally the strength of lean mixes is increased and the strength of rich mixes is decreased.

Examples of materials used in amounts ranging from 10 to 30 percent of the weight of the cement are: fly ash, volcanic ash, heat-treated diatomaceous earths, and either heat-treated or raw shales or clays.

Further discussion of the theoretical considerations involved in the use of pozzolanic materials and mineral powders is included in the section "Theoretical considerations on the use of pozzolanic materials and mineral powders as admixtures." A discussion of certain pozzolanic materials which can be used in reducing the alkali-aggregate reaction will be found in the following section.

ALKALI-AGGREGATE EXPANSION INHIBITORS

General

Use of admixtures for the specific purpose of counteracting alkali-aggregate reaction is of rather recent development, and for this reason the material presented here should be considered only as a brief resumé of the most recent knowledge of the subject.

The alkali-aggregate reaction, which has been attributed to the interaction of the alkalies in portland cement and certain siliceous types of aggregates, may result in excessive expansion and pattern cracking of concrete. Deterioration of concrete due to this reaction has been variously described in the literature.[21-26] The term alkalies as applied to portland cement refers to the sodium and potassium compounds present in relatively small proportions expressed as the oxides. When this particular type of expansion phenomenon was first described by Stanton in 1940,[27,28] the only apparent remedy or palliative was the use of portland cement of low-alkali content (0.60 percent or less computed as Na_2O).

To date, field experience and service records of concrete in which cements of low-alkali contents and reactive aggregates were employed have not indicated that excessive expansions will occur as the result of alkali-aggregate reaction. However, a considerable number of laboratory tests have indicated that destructive expansions, as the result of alkali-aggregate reaction, are possible with some aggregates when used with comparatively low-alkali cement.[29,30] It has also been ascertained that certain natural or artificial pozzolans are capable of reducing the expansion caused by high-alkali cements when used in combination with reactive siliceous aggregates. It should be noted that those materials found to be effective in counteracting the alkali-aggregate reaction are pozzolanic materials. However, the converse is not necessarily true, some proven pozzolans showing little ability to prevent expansion. For this reason, it is necessary to have available some relatively rapid means of evaluating the efficiency of a given material for the purpose intended. At the present time, the only simple and rapid method of evaluating the ability of pozzolanic materials to control alkali-aggregate reaction is an accelerated reduction-in-expansion test.[31,32]

Types of effective pozzolans

Pozzolans which have been found to reduce the expansion caused by alkali-aggregate reaction in concrete or mortar may be classified in three categories. The first covers materials containing "amorphous" siliceous or siliceous and aluminous substances and includes some opals and highly opaline rocks, certain volcanic glasses, diatomaceous earth, and calcined clays of the kaolinite type. Also included in this category are fly ashes, some of which significantly reduce the expansion caused by alkali-aggregate reaction and others which do not. The second category covers calcined clays of the montmorillonite type. The effective montmorillonite type clays must not contain calcium as the exchangeable cation and they must be calcined in the 1000 to 1800 F range, but not at a temperature sufficiently high to destroy the crystalline structure. The third covers combinations of the above two categories and includes such materials as siliceous shales; *e.g.*, Monterey shale and altered pumicites which are mixtures of montmorillonite clays and volcanic glasses. A more detailed classification of pozzolans based upon mineralogic composition has been published recently.[33,34] One reference[34]

contains also a discussion of natural pozzolans from the standpoint of geologic occurrence.

The amount of suitable pozzolan required in a concrete to control this reaction will vary with individual aggregates and alkali content of the cement. It has been found that ample protection can normally be obtained by quantities ranging from 20 to 35 percent by weight of the cement. These amounts are usually compatible with other physical requirements of the concrete such as strength, workability, and other properties. The foregoing should, however, be considered as a generalization since it has also been ascertained that certain material when finely divided and of high opal content (*e.g.*, certain diatomaceous earths and opaline cherts) will prevent expansions when used in amount of less than 15 percent by weight of the cement. Consideration must also be given in the use of these latter materials to the usually attendant increased water requirement.

Other admixtures

There is some evidence that air-entraining admixtures reduce slightly the expansion caused by alkali-aggregate reaction. There is also some evidence that aluminum powder reduces this expansion slightly as the result of gas entrainment. Recent test data indicate that small additions of certain other chemical substances may hold promise as inhibitors of alkali-aggregate expansion.[35] Outstanding reductions in expansion of laboratory mortar specimens have been reported for additions of 1 percent by weight of the cement of lithium salts. Moderately good results were also obtained with certain protein air-entraining agents. It was found that some of these latter substances were more effective in reducing expansion than others, even when the amount of air entrained was the same. The results reported are limited and further confirmatory work is needed with regard to these admixtures. Laboratory tests indicate that the use of calcium chloride increases the expansion caused by the alkali-aggregate reaction.

Conclusions

In connection with the use of an admixture to counteract alkali-aggregate reaction, the question can logically arise whether an effective material is sufficient to counteract this reaction *per se*. That is, could a high-alkali cement be used with a highly effective admixture? Also, the question can be asked, can a less efficient admixture, from the point of view of the alkali-aggregate reaction, be used with a low-alkali cement or with less active aggregates? Until these points can be resolved, it is the opinion of the committee that only the most efficient or proven materials should be used in conjunction with low-alkali cement where reactive aggregates are of necessity to be employed in the work.

DAMPPROOFING AND PERMEABILITY REDUCING AGENTS

General

Water in liquid form passes through concrete by two different processes. In one process, water under pressure and in contact with one surface of the

concrete is forced through either relatively large or small channels connecting the inner and outer surfaces. Permeability measurements under sizable pressures give an indication of the ability of a concrete to resist this type of flow. Use of an admixture or other process to render concrete more resistant to this type of flow is termed in this report "reduction of permeability." It is the opinion of the committee that this term is more appropriate than the rather commonly accepted term "waterproofing." In the second process, the passage of moisture through concrete may take place in the absence of any appreciable external water pressure simply by the action of capillary forces which draw the water into and throughout the mass. Evaporation from surfaces exposed to unsaturated air and the constant replenishment of moisture from the surface in contact with water result in a flow of moisture through the concrete. The reduction of moisture transfer of this second type is termed "dampproofing." Measurements of absorption and capillary rise as well as of permeability under low pressures give an indication of the dampproof qualities of a concrete. Some admixtures may have merit for both reduction of permeability and dampproofing.

Many investigators are of the opinion that concrete, carefully designed, placed, and cured, will be impermeable without the use of any admixture, and that leakage due to cracks, joints, and honeycombing is apt to be of much greater magnitude and concern than leakage through the sound mortar. However, since the usual design, placing, and curing leave much to be desired, the use of a well-chosen admixture may prove advantageous where low permeability is required, especially for concrete made from a lean mix.

Even though the best concrete practices are adhered to and the resulting concrete is essentially impervious to water under pressure, passage of moisture due to capillary action may take place if the concrete is in contact with moisture or damp earth. Moisture movement of this type may often be responsible for unsightly stains on concrete and masonry walls, efflorescence, excessive humidity in adjacent air spaces, and other objectionable results of dampness. Often the moisture is evaporated and carried off by ventilation at such a rate that the concrete appears dry, but the relative humidity of the surrounding air may be higher than desirable.

Water repellents employed as integral admixtures are effective in reducing the rate of capillary transmission and in many cases offer the simplest method of controlling such moisture movement. They are used not only in concrete walls and slabs but also in cast stone, ornamental concrete, stucco, cement water paints, and pargings. They are not effective against sizable water pressures.

Use of such admixtures should in no case be considered as a substitute for careful choice of materials and mixes, for skilled workmanship, or for adequate curing. In no case can an admixture be expected to compensate for cracks or large voids in the concrete, although it may minimize the probability of their occurrence.

Since the effectiveness of an admixture in a given application depends upon many factors, including not only the placing and curing of the concrete but also the properties of all the individual materials and their proportions, the choice of an admixture in a particular instance should be made only after experimental work has been conducted or previous experience with similar applications has shown that beneficial results will occur.

Many preparations are sold under the name of integral "waterproofers" or dampproofers. They vary widely in composition and in nature of their action.[36,37,38] Specific preparations are not discussed in this report but the various general types of integral materials are considered. They are discussed from the standpoint of composition of the additive, of effect on strength and other physical properties of concrete, and of probable effectiveness as permeability-reducing or dampproofing agents.

Calcium chloride solutions

This portion of the report applies only to calcium chloride solutions, not to the many commercial preparations of calcium chloride containing small quantities of other materials which will be considered later.

The listing of calcium chloride as a dampproofing or permeability-reducing agent has been questioned. However, it is included as such because a number of proprietary materials sold as so-called "waterproofings" consist of little more than a calcium chloride solution. Calcium chloride's most important function is that of an accelerator, serving to increase the rate of cement hydration and of strength development and is fully described in the section "Accelerators," p. 116. Little difference is found in the permeability or absorption of similar concretes with and without calcium chloride.

Under certain conditions the use of calcium chloride may prove beneficial. The importance of adequate curing in the making of impermeable concrete is well recognized. Use of an accelerator shortens the period during which curing must be continued for best results. This fact should not be construed as a recommendation that employment of an accelerator makes good curing unnecessary but rather that the effects of bad curing practices will be lessened.

Soaps

Admixtures classified here as "soaps" are inorganic salts of fatty acids, usually calcium or ammonium stearate or oleate. In commercial preparations the soap content is usually 20 percent or less, the balance of the solid material being lime or calcium chloride. The quantity of soap incorporated in concrete should not exceed 0.2 percent by weight of the cement. Larger percentages result in excessive frothing action with consequent increase in void space and decrease in strength. From the standpoint of dampproofing of concrete, a soap functions primarily as a water repellent. Its relative effectiveness in this respect is dependent upon its uniformity of distribution throughout the concrete.

Tests[39] have shown that, in general, concretes into which soaps have been incorporated are more permeable to water under pressure than similar ones

without such treatment. This has been attributed to the foaming action noted during mixing, but the explanation is not wholly consistent with permeability results obtained with air-entrained concrete. It should be mentioned that soaps frequently improve the workability of concrete, contributing to greater ease of placement. Their use may thus be beneficial in that they lessen the probability of the formation of large voids or pockets.

Use of calcium chloride, in conjuction with soaps, tends to compensate for the reduction in concrete strength at early ages. Ultimate strengths, however, are normally lower than for plain concrete. A word of caution should be given with respect to curing procedures for concrete containing this type of agent. Intermittent curing cannot be used because once the concrete has dried out it cannot readily be wetted again.

Butyl stearate

The action of butyl stearate in concrete is similar to that of the soaps. Unlike the soaps, it does not have a frothing action with the concomitant decrease in density of the concrete. It may be used, therefore, in much greater quantity than the soaps. To attain greatest uniformity of distribution throughout the concrete it is usually added as an emulsion. A quantity of emulsion found suitable is that which gives the equivalent of 1 percent of butyl stearate by weight of the cement.

Butyl stearate is superior to the soaps as a water repellent in concrete. It is noticeably more effective against penetration of water by capillary action when under moderate exposure, and it does not increase the permeability as the soaps frequently do. When used in recommended amounts its effect on the strength of concrete is negligible.

Finely subdivided dry materials

Materials in this category may either be inert or may react with other constituents of the mix. Reactive materials are usually pozzolanic, combining with lime liberated during hydration of the cement.

The reduction in permeability achieved by the use of finely subdivided dry materials is closely related to the properties of the particular concrete mixture. If the cement content of the cement-water paste is relatively low or if the aggregate is deficient in fines, the addition of the finely subdivided material will probably be highly beneficial, decreasing permeability and increasing strength. However, if the concrete mix is rich in cement and the aggregate properly graded, added fine material will probably be detrimental since the increased surface area increases the water demand resulting in a less dense and a weaker concrete. The above comments apply whether the added materials are inert or reactive. Regardless of mix quality, reactive materials have an advantage over the inert materials in that some additional strength results from the reaction and laboratory tests have demonstrated that many pozzolans are effective in substantially reducing permeability.

An increase in cement content is often more effective in decreasing the permeability of concrete than is the addition of other fine material. In

massive structures, the desirability of maintaining heat evolution and subsequent shrinkage at a minimum requires that the cement content be kept low. In such cases, finely subdivided materials, preferably pozzolanic in character, may be used advantageously. Also, in any structure, an excess of fine material, including cement, may lead to objectionable properties of the concrete and should not be resorted to as a substitute for proper grading of the aggregates. For further discussion of the theoretical aspects of this subject, reference is made to "Theoretical considerations on the use of pozzolanic materials and mineral powders as admixtures," p. 139.

Tests have shown that finely divided materials have little merit as dampproofers.

Petroleum products

Heavy mineral oil has proved effective in rendering concrete water repellent. The oil should contain no fatty or vegetable oil components and should have a viscosity of approximately SAE 60.

The use of types RC-2 and MC-0 asphalt cut-back oils have been tried in quantities of 2½, 5, and 10 percent by weight of cement. Strength and workability of the concrete was not seriously affected although it was reported that 10 percent oil increased the setting time by approximately one-fourth.

Asphalt emulsions are also used. Such materials are not effective until the emulsion has been broken by the drying of the concrete. Two-in. laboratory cubes made with such a material and dried, after curing, at 65 C showed a pronounced reduction in 1-hr and 24-hr absorption. Twenty-eight day strengths were reduced by less than 10 percent.

Workability agents and air-entraining admixtures

Since the production of concrete of low permeability depends to a great extent on successful uniform placing of the material, an agent which improves the plasticity of a given mixture without causing deleterious effects or which limits bleeding and thereby reduces the number of large voids might properly be classified as a permeability reducing agent.

Certain organic compounds or mixtures have been marketed as water-reducing agents under proprietary names. In addition to increasing workability, the sulfonated type generally entrains some air and decreases the bleeding, while carbohydrate salts in combination with other active or inert materials affect the workability favorably but do not cause air entrainment or reduce bleeding.

Air-entrained concretes have greater plasticity and bleed less than those without entrained air. The reduction in number of large voids left by poor compaction or caused by collection of bleeding water under aggregate results in less permeability. The view has been expressed that in addition to acting as a workability agent the small disconnected voids offer a barrier to the capillary passage of water by breaking the capillaries. Air-entrained concretes have lower absorption and capillarity than those without air and hence air entrainment may be classed as both a permeability reducing and dampproofing agent.

Miscellaneous materials

Some proprietary powders, pastes, and liquids available on the market as so-called integral "waterproofers" cannot be classified under types thus far discussed. These products are made up of various combinations of materials such as (1) barium sulfate, calcium and magnesium silicates, and fatty acid; (2) finely ground silica and napthalene; (3) colloidal silica and a flousilicate; (4) petroleum jelly and lime; (5) cellulose materials and wax in an ammoniacal copper solution; (6) silica, lime, and alum; (7) coal tar cut with benzene; and (8) sodium silicate together with an organic nitrogenous material.

The chemical or physical actions of these miscellaneous materials on concrete are varied. Their effect on the strength of a properly designed concrete is usually detrimental. The effectiveness for reduction of permeability of those containing water repellents, finely subdivided materials, or oils can be estimated from the foregoing discussions of such materials. Based on available test data, none of the miscellaneous materials enumerated above are appreciably effective in dampproofing concrete.

WORKABILITY AGENTS

General

The ease with which concrete can be placed influences the cost of concrete construction.[49] The workability of concrete should be the maximum obtainable commensurate with desired economy. In the following instances, however, increased workability is necessary almost without regard to increased cost: (a) if the concrete is harsh because of aggregate characteristics or grading; (b) if the concrete must be placed around closely spaced reinforcement or in difficultly accessible sections; (c) where special means of placement are required such as with tremie or pumping methods. It should be noted that frequently reproportioning of the mixture or increasing the cement content may give the desired results.

Finely divided materials

In mixes deficient in "fines" (particularly material passing the No. 200 sieve) addition of a finely divided mineral admixture improves workability, reduces the rate and amount of bleeding, and increases the strength. In general, the higher the specific surface of the material the smaller the volume required to produce a given effect on workability. When an appropriate quantity of mineral powder is used, no increase in total water content of the concrete is required and drying shrinkage and absorptivity of the hardened concrete are not much affected. Addition of a mineral powder to mixtures not deficient in fines, particularly mixtures rich in portland cement, generally decreases workability for a given water content. For that reason, the addition of mineral powders to such mixes generally entails an increase in the total water content of the concrete and may result in an increase in drying shrinkage and absorptivity and a decrease in strength. A discussion of the theoretical

aspects of this subject is contained in the section "Theoretical considerations on the use of pozzolanic materials and mineral powders as admixtures," p. 139.

Examples of materials added in amounts not exceeding 3 to 5 percent by weight of the cement are bentonite, clay, and diatomaceous earth. Examples of materials added in larger amounts are fly ash, finely divided silica, fine sand, hydrated lime, talc, and pulverized stone.

Water-reducing agents

Certain organic compounds or mixtures that have been marketed under proprietary names increase the slump of concrete of a given water content and, therefore, permit concrete of a given slump to be produced at a somewhat reduced water content. Only a few such materials have been examined; hence, no broad statements can be made of the general effects of all material in this group. The materials that have been examined to date, however, contain either sulfonated organic compounds or carbohydrate salts in combination with other active or inert materials. The sulfonated type generally caused some entrainment of air and decrease in bleeding, but not to the extent caused by materials classed as air-entraining agents. This type had a proportionately greater effect in lean mixes than in rich ones. The carbohydrate type did not cause entrainment of air and did not reduce bleeding; some retardation of set was produced.

The effect on the hardened concrete of the use of these materials, when advantage of the possible water reduction was taken, was generally somewhat improved impermeability and compressive strength and, for the type causing mild air entrainment, improved resistance to freezing and thawing.

As with other chemically active admixtures, the specific effect of these materials varies with different cements and tests should be made with the job cement and preferably the other job materials before their use is attempted.

Air-entraining agents

Although these admixtures were originally considered for use principally because of their effect on the durability of concrete, their effect on the workability of leaner mixes is so pronounced as to merit mention in any discussion of workability agents. The incorporation of numerous small well-distributed air bubbles in the concrete acts in the manner of a lubricating medium, generally increasing the "fatness" of the mix and markedly improving the placeability of otherwise harsh concrete. As noted in the section on air-entraining agents, p. 120, entrained air improves cohesiveness and accordingly reduces segregation, reduces rate and amount of bleeding, and permits a reduction in mixing water without loss of workability.

GROUTING AGENTS

General

Many admixtures will contribute to the maintenance in suspension of the cement particles until the hydration process is well underway. Bleeding in grout is objectionable and must be minimized by the use of some admixture that possesses the necessary characteristics to produce a grout that can be

satisfactorily pumped over long distances at a lower water-cement ratio than is possible with plain portland cement grout.

Too much water in grout is just as objectionable as too much water in concrete. Dense, impervious, durable grout, the kind that will not permit the slow infiltration of water and the consequent leaching out of the cementing medium, can only be produced with relatively low water-cement ratios, ratios much lower than now ordinarily used. Such low water-cement ratios are possible only when the grout mixture includes, in addition to the portland cement and water, a suitable admixture that will "lubricate" the mass, and thus make possible the injection of drier grout mixtures at reasonable pressures and without segregation of ingredients. One of the basic shortcomings of grout mixtures composed of portland cement and sand is the tendency of the mixture ingredients to separate during the placing process.

In grout containing sand, entrained air is beneficial in reducing friction during placement. Too much extremely fine inert material inhibits the beneficial effect of air entrainment. Without sand in grout no beneficial effect can be obtained by entraining air and hence some other solution must be substituted for air entrainment wherever sand is not a part of the grout mixture. When grout is pumped into subaqueous prepacked gravel or any other porous foundation material below the water table, it is mandatory that the ingredients do not segregate as it is deposited under water. Admixtures that will minimize segregation under water are likewise available on the market. Any project on which an extensive grouting program is contemplated should set up a core-drilling and grouting investigation to develop the proper technique for that particular work and to try out various admixtures. There is no other way of satisfactorily solving such problems.

Admixtures used for grouting

Only general recommendations on the use of admixtures can be given because of the diversified problems encountered in grouting and because of the lack of information on field use. Pilot tests must be made prior to using any admixture in a grout because some admixtures act differently with different types and brands of cement. Admixtures are classified in the following paragraphs according to their effects on the properties of the grouting mixture.

Accelerators—Accelerators may be used in a grout to hasten the set in situations where a plugging effect is desired. Calcium chloride or triethanolamine can be used. The latter is more effective with even small amounts causing almost instantaneous set. The use of accelerators is further described on p. 116 of this report.

Retarders—Retarders and dispersing agents can be used in a grout to aid in pumpability and in the penetration of grout into fine cracks or seams. Through their dispersing nature they slow down the coalescing of the cement particles. This, in effect, also reduces the amount of water required for the same fluidity. Included in this category are RDA, SRDA, mucic acid,

Plastiment, and Intrusion Aid. The use of retarders is further described on p. 118 of this report.

Gas-forming agents—Gas-forming admixtures can be used when grouting in completely confined areas, such as under machine bases, in contraction joints, or in foundations. Aluminum powder is the agent most commonly used and it reacts to produce small bubbles of hydrogen gas throughout the mixture. This reaction produces expansion of the grout and eliminates settlement with the resultant complete filling of the cavity. The use of gas-forming agents is further described on p. 124 of this report.

Workability agents—Such materials as fly ash, bentonite, pumicite, and diatomaceous earth are sometimes added to improve pumpability, reduce settlement, and provide increased economy. Fly ash has a pozzolanic effect and it aids pumpability, retards setting time, and, in some areas, is less expensive than cement. Bentonite aids the retention of the cement particles in suspension during the pumping operation and increases the yield of the hardened grout. Pumicite and diatomaceous earth also have a pozzolanic effect and in small amounts, because of their high water requirement, will make a grout thicken and be more cohesive or gelatinous. Where voids are large enough to allow the use of sand-cement grout, the addition of the sand will reduce the cost of the grout mix. With the use of other admixtures and with the proper amount of water, the sand-cement grout can be readily pumped. The use of workability agents is further described on p. 134 of this report.

Admixtures for oil well grouting

The use of retarders in oil well grouting is discussed on p. 119. In some cementing operations it is necessary to use a cement grout which will not lose its water content easily to the walls of the well. Lower water loss cements have been prepared by the addition of gels and clays, soya bean protein and sodium hydroxide, pre-gelatinized starch, methyl cellulose, or mixtures with clay and ferric oxides. In certain types of wells the oil bearing formations are sealed off from the well by cement which is later perforated by bullets shot through the cement casing. Cracking of the concrete shell may be lessened by the reduction in brittleness brought about by the addition of bentonite clays in an amount of about 6 lb per bag of cement.[41]

MISCELLANEOUS

Unclassified

There are a number of commercial admixtures which do not fit into any of the foregoing classifications. This group includes materials which are claimed to increase the resistance to wear or abrasion (integral floor hardeners) or to react with the cement or the products of hydration of the cement, filling pores, thus decreasing absorption and permeability. These materials are difficult to evaluate but in no case can they be considered a cure for poor workmanship.

Expanding cements

Expanding cements are not necessarily admixtures and should not logically

be so classified. However, since their effects and uses are closely related to those of admixtures, it has been felt desirable to describe them under the "miscellaneous" classification.

Concrete shrinks when it is dried out and the disadvantages of this shrinkage include the cracks which may result and expose the interior and its reinforcing steel to aggressive agents in the atmosphere or water. Shrinkage may cause undesirable internal stresses, less effective use of reinforcing, and poor contact in machinery grouting operations and patching work. Shrinkage may be reduced by proper proportioning of the mix and its bad effects minimized by adequate curing. But complete elimination can be realized only by adding an opposing expanding element to the concrete which will produce an expansion equal to the normal shrinkage during each time interval. Such an ideal material has not been found and in practice an attempt is made to produce an expansion during the damp hydrating period which will balance the subsequent drying shrinkage. Such a cement is called a nonshrinking cement while one in which the initial expansion is greater than the drying shrinkage is called an expanding cement. Obviously, the difference is a matter of degree.

In Europe, expanding cements have been produced by the addition of a sulfoaluminous cement made by burning a mixture of gypsum, bauxite, and limestone.[42] This cement, which is stable by itself, when added to portland cement, itself stable, produces an expansion. The amount of expansion is controlled by varying the proportions used and by adding ground slag which acts as a stabilizer by absorbing the excess calcium sulfate. The time at which expansion takes place is controlled by varying the quantity of slag and the fineness of both the slag and sulfoaluminous cement. The latter must be coarse enough so that appreciable premature expansion will not lower the initial strengths. The slag must be coarse enough to allow the desired expansion before appreciable stabilizing takes place. Obviously, the curing condition and length of moist curing may affect the results obtained. Amounts of sulfoaluminous cements from 9 to 25 percent by weight of the portland cement are used and a slag content of 15 to 20 percent of the total has been found to give satisfactory results.

Powdered iron plus an oxidizing agent produces the expansion in another type of nonshrinking or expanding cement. In this case, as previously discussed, control of the amount of expansion is also difficult. A concrete with an iron admixture may be expected to expand some under every subjection to moisture so that while the initial expansion might be of the correct amount to compensate for the subsequent drying shrinkage, later "growth" in the presence of moisture might prove excessive. A number of proprietary admixtures and cements of this type are on the market.

Grinding aids

Grinding aids are not admixtures and accordingly do not logically fall within the scope of this classification. By reason of their widespread use, however, and because of this indirect effect on concrete a brief discussion of them is

given. Use of grinding aids has contributed to the finer grinding of cements. This in turn has led to higher early strengths or leaner mixes that were placeable and which yielded adequate strengths. Materials such as Vinsol resin and tallow, in amounts of the order of 0.03 to 0.06 percent, are aids to grinding, but are generally used for the subsequent direct effect as air-entraining agents in concrete rather than for their effect upon grinding. Water, coal, and rosin, are the oldest used grinding aids. The commercial grinding aid TDA when added in the amounts used as a grinding aid (limited by current ASTM specifications to 0.043 percent by weight for Type I cement and 0.08 percent for Type III, high early strength cement) and the commercial material "109-B" (limited by current ASTM specifications to 0.03 percent by weight for Type I cement and 0.05 percent for Type III cement) are generally considered to have only little direct effect on concrete.

THEORETICAL CONSIDERATIONS ON THE USE OF POZZOLANIC MATERIALS AND MINERAL POWDERS AS ADMIXTURES

Admixtures have been used to improve the workability and bleeding characteristics of fresh concrete and sometimes to modify the properties of the hardened concrete. Many of the materials so used are mineral powders as fine as or finer than portland cement. They therefore serve to "correct" the properties of the paste. Since the distinction between the fine material in the paste and that in the aggregate is more or less arbitrary, mineral admixtures may also be considered as correctives for aggregate gradation.

Some mineral powders are relatively inert chemically, others are pozzolanic or cementitious. Their chemical characteristics are of secondary importance so far as the effect on workability and bleeding characteristics of fresh concrete is concerned.

Bleeding and plasticity

Among the various factors determining the bleeding characteristics and the degree of plasticity of fresh concrete, the amount of surface area of the solids per unit of water volume is the most important.[43] When the ratio of surface area to water is too low, the paste has a thin, watery consistency. Consequently, at a given slump the volume of paste in the concrete only slightly exceeds the volume of voids in the aggregate. This means that the aggregate particles are only slightly separated by paste layers and hence that the mix has a low capacity for plastic deformation and a high tendency to segregate.[44] It denotes also a high incidence of particle interference according to Weymouth's criterion.[45]

When the ratio of surface area to water is too low, the rate of bleeding is relatively high. Moreover, most of the bleeding does not appear at the surface; that is, the aggregate particles settle for a short period and then establish point-to-point contacts that prevent further settlement. The watery paste continues to bleed within the pockets defined by the aggregate particles leaving layers of water at the undersides of the particles. Thus,

with mixtures having the characteristics just discussed, bleeding tends to reduce the homogeneity of the concrete.

Undesirable effects of bleeding

In extreme cases, the lack of homogeneity is manifested by open fissures under the aggregate particles large enough to be visible in a cross section of the concrete. As early as 1917 Edwards[46] published photographs of cross sections showing the separation of the paste from the undersides of the aggregate particles. In 1927 (and at various later times) Gilkey[47] discussed this phenomenon. In 1929 Powers[48] published experimental evidence of its adverse effect on strength and watertightness. Collier[49] in 1930 referred to this phenomenon in explaining deviations in strength at constant water-cement ratio. Ruettgers, Vidal, and Wing[50] (1935) showed that at a given water-cement ratio and stage of curing the concrete was more permeable the larger the maximum size of the aggregate. These results seemed to demonstrate that the fissures under the aggregate particles contribute in a major way to the permeability of the concrete. Casey[51] (1937) published photomicrographs showing fissures under the aggregate particles for concretes made with cements having different bleeding characteristics. In 1939 Powers published experiments showing that the loss in strength at a given water-cement ratio was a function of the degree to which bleeding is hindered by particle contacts.

Particle separation

The undesirable effect of bleeding described in the foregoing paragraphs can be ameliorated by increasing the ratio of surface area to water in the paste. This generally increases the stiffness of the paste and at a given slump effects a wider separation of the aggregate particles in the concrete. The ratio of surface area to water volume may be increased by increasing the amount of cement (lowering the water-cement ratio of the paste) or by supplementing the cement with a suitable admixture.

However, the degree of particle separation required to eliminate undesirable bleeding effects is high. It requires the volume of the mix to exceed that of the dry rodded aggregate by as much as 50 percent. To effect such a degree of separation requires a paste of very low water-cement ratio or the use of an excessive amount of admixture. It is necessary, therefore, to base the selection of materials and proportions on a compromise; some potential strength and impermeability must be sacrificed to obtain lower water contents and consequently better volume-change characteristics.[52]

Although under some circumstances the use of admixtures lowers the amount of voids in the concrete, admixtures should not be regarded as void-fillers—at least not in the sense of providing small particles to fill the voids between the larger particles. A mixture of either cement and water or cement, admixture, and water forms a soft, plastic paste—technically a weak solid which envelops the aggregate particles and, in the course of the mixing process, separates them. Thus, in a sense, the function of the admixture is to

increase the voids in the aggregate and in so doing to increase the capacity of the mix for plastic deformation. Adding 1 percent of a material of high specific surface such as bentonite together with the necessary amount of water will effect the same increase in aggregate-particle separation as 10 or 15 percent of a mineral powder of ordinary specific surface. This shows that the volume of paste produced by the admixture rather than the manner in which the particles fit into the voids of the aggregate is the factor controlling the effectiveness of the admixture.

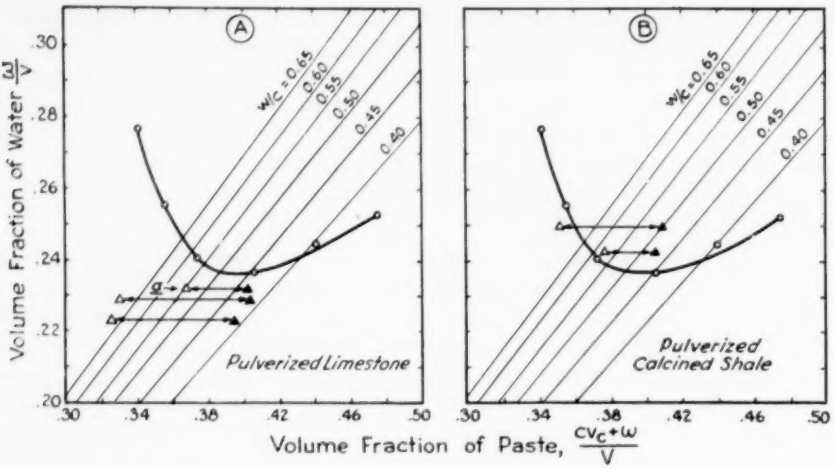
In 1931, G. M. Williams[44] reported: "The relative efficiency of admixtures in increasing workability is measured by the relative volumes of pastes produced when equal weights of admixtures are made into pastes of approximately equal flowability." Practically the same conclusion was published in 1932 by Powers,[53] who reported data indicating that the workability of concrete was determined by the quantity and consistency of the paste rather than by the characteristics of the powder or powders of which the paste is made. This study indicated also that for any given material there is an optimum paste consistency at which the water requirement of the concrete for a given mobility is at a minimum. In still later studies McMillan and Powers[54] in reporting a study of admixtures stated: "Any of the powdered materials tested, or blends of two materials, when used in such proportions as to produce a paste of given consistency, gave, when combined with the same gradation of aggregate, concrete mixes of substantially equal workability." A method was given for computing the required amount of admixture on the basis of the water content of admixture-pastes tested separately. Such computations serve as a first approximation to be checked by trial batches. John C. Sprague[54] also found that the relative amount of admixture required to control the bleeding capacity of concrete was roughly proportional to the relative amount of water required by the admixture for "normal" consistency.

Selection and proportioning of admixtures

From the point of view presented in the preceding section, selecting and proportioning admixtures should be based on the optimum paste consistency which will produce concrete of specified consistency with the least amount of mixing water. Paste-making characteristics of the admixture are discussed in the following. The object is to produce a paste having optimum consistency and having, potentially, the strength specified.

Fig. 1 shows a typical relationship between water content and paste content, paste being defined as the sum of the volumes of water and cement. Since the values plotted on the scale of abscissas differ from those on the scale of ordinates only by the volume of cement present there is a simple mathematical relationship between water-cement ratio, water content, and volume of paste, as shown by the slanted lines.* The locus of the plotted points represents a relationship established by making a series of concrete

* $w = \left(\frac{1}{1 + \frac{v_c}{W/C}}\right)(cv_c + w)$



c = weight of cement
v_c = sp. vol. of cement
w = volume of water
V = volume of concrete

△ - water vs. cement paste in a mix containing admixture
▲ - water vs. cement-admixture paste

Elgin sand, fineness modulus 2.90
Remolding Effort, 43 jigs

Fig. 1—Typical interrelationships of water content, water-cement ratio, and paste content for mixes with and without admixtures

mixes of the same cement and aggregate, each mix having the same consistency, as indicated by the remolding test. (Similar relationships can be established on the basis of the consistency as given by the slump test, or as judged by an experienced operator.)

For these materials the minimum water content was obtained with a paste content equal to 40 percent of the volume of the mix. The consistency of the paste represented at this point is therefore the optimum paste consistency for these materials and selected concrete consistency. It is the consistency of the neat paste having a water-cement ratio of 0.45 by weight.

In selecting and proportioning an admixture for concrete made with this cement and aggregate, the designer assumes that the paste should have the same consistency as the neat cement paste with water-cement ratio 0.45, regardless of the make-up of the paste. Therefore, if the concrete required a water-cement ratio of 0.45 or lower, no admixture would be specified.* If a water-cement ratio higher than 0.45 is desired, an admixture could be used advantageously. For example, if the desired water-cement ratio is 0.55, a plain mix would, as shown in Fig. 1A, have a paste content of about 36.5 percent instead of the desired 40 percent. Therefore, a combination

*In case a W/C less than 0.45 is required, the consistency of the paste will of necessity be stiffer than optimum.

of cement and other mineral powder having the same consistency as a plain cement paste of $W/C = 0.45$ would be sought.

The open triangle *a* and the solid triangle to which it is tied in Fig. 1A illustrate how this was done with pulverized limestone as an admixture. The open triangle indicates the water-cement ratio of the cement paste and the solid triangle gives the volume of the augmented paste. Thus the desired result was accomplished: the water-cement ratio was 0.55, the paste fraction was 40 percent, and the consistency was as specified in terms of remolding effort.

The total water content of the mix made with the limestone-cement paste was slightly below that of the optimum mix made with cement alone. This indicates that at a given paste consistency the solid content of the cement-limestone paste was greater than that of the neat cement paste. This desirable reduction in water content is not always to be expected. Fig. 1B shows the results of using pulverized calcined shale instead of pulverized limestone. With this material the cement-admixture paste had a lower solid content than neat cement paste of the same consistency. Consequently, concrete made with cement and pulverized calcined shale had a higher water content for the same paste content and paste consistency than did the optimum plain mix.

In general, the higher the proportion of solids in a paste of given consistency the lower the water content of the concrete at a given degree of plasticity. If a cement-admixture paste contains the same volume of solids as neat cement paste of the same consistency, the use of the admixture in correct proportion will have little or no influence on the water requirement of the concrete. If the solid content of the cement-admixture paste is lower than that of the cement paste having the same consistency, using an admixture is liable to increase the water requirement of the mix. It follows therefore that admixtures giving pastes of relatively high solid content are to be preferred.

ACKNOWLEDGMENTS

The committee wishes to stress the point that a considerable amount of the material contained in this report has been taken from the original November, 1944, Committee 212 report on admixtures. For this reason we wish, as an acknowledgment of this earlier work, to publish the membership of the committee at that time: F. B. Hornibrook, chairman, R. F. Blanks, R. E. Davis, A. T. Goldbeck, F. H. Jackson, T. C. Powers, and B. W. Steele.

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Title No. 51-6

Precast Concrete Sandwich Panels for Tilt-Up Construction*

By F. THOMAS COLLINS†

SYNOPSIS

Design and construction details are presented on precast concrete wall panels of sandwich panel design. While most precast concrete sandwich wall panels to date have been factory fabricated in relatively small panels, advantages are cited for casting large sandwich panels for tilt-up construction. Details are given on various types of sandwich panels. Some fundamentals of design for conventional sandwich walls are given with sample calculations. An example wall is designed for tilt-up construction. Incorporation of prestressing in such a panel is discussed. A cost analysis is also included.

INTRODUCTION

By definition, precast concrete sandwich construction consists of two faces of relatively thin, high strength, high density materials bonded to a core of relatively thick, low density material. The function of the core material is to stabilize the relatively thin faces of precast, high strength concrete and to provide a high stiffness factor for the combination of materials by separating the faces. This combination produces a lighter, stronger wall, and if the core material is a good insulator it produces a more insulated wall.

Although sandwich design is considered relatively new to the construction industry, there is evidence that the sandwich principle was used as early as 1849 by William Fairbairn in experimenting with bridge design using laminated wood decking and concrete as a composite beam.

As early as 1906 in the United States the possibilities of constructing sandwich concrete walls was intriguing designers and contractors. In that year a building was constructed with "sandwich" tilt-up wall panels. The platform on which the front was cast was tilted with the wall (Fig. 1). Some of the walls were hollow, made by casting a 2-in. layer of concrete, then a 2-in. layer of sand, then a 2-in. layer of concrete. The two outer layers of concrete were tied together with ties or reinforcement; the exact design is unknown. The sand between the two outer faces of concrete was washed out with water as the wall was lifted.

In 1933 Swedish builders made a sandwich wall of 5-in. lightweight concrete block cast and bonded to dense in situ concrete (Fig. 2a). A $\frac{5}{8}$ in. thick

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Fig. 1—Early example of tilt-up "sandwich" construction. Some of the walls were hollow, made by casting a 2-in. layer of concrete, placing a 2-in. layer of sand, and then casting the top 2-in. layer of concrete. The sand was washed out with a fire hose as the wall was tilted

lime cement mortar was then plastered on the outside of the lightweight concrete block. This wall gave excellent results. It has also been common practice in cold storage work in the United States to place the concrete wall in situ, seal the inside face, apply the required thickness of insulation, and then plaster the inside face (the cold face) with a 1-in. cement plaster (Fig. 2b). This is more or less sandwich wall panel construction.

Other sandwich walls have used cored gypsum filler block as the insulation material. E. I. du Pont de Nemours Co. used this type of precast panel in 1946 on industrial buildings. There have also been several other systems developed using cores of cellular glass, lightweight concrete, and other insulating materials.

SANDWICH SLAB CONSTRUCTION

More recent examples of sandwich slab construction include both "open-face" and "double-face" construction. The examples that follow are only a few of those that have been developed.

Open-face sandwich panels

In the open-face type, the wall consists of just two layers of material—

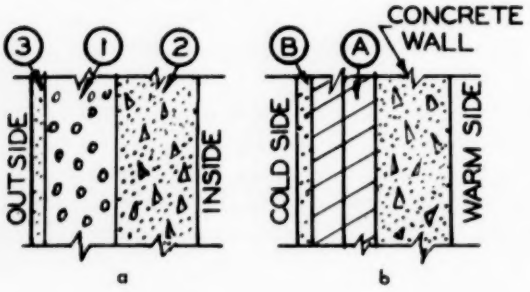

Fig. 2a (left)—Swedish sandwich wall: (1) 5-in. lightweight concrete block; (2) cast-in-place dense concrete wall; and (3) ⅝-in. layer of lime cement mortar. Fig. 2b (right)—American insulated wall used in cold storage buildings: (A) insulation and (B) 1 in. thick cement plaster

a hard outer layer and a second soft insulation layer. The two examples of open-face sandwich construction reported here used vermiculite concrete as the insulating layer. Since the vermiculite concrete did not provide enough structural strength when used alone, the dense concrete layer was placed down first as the structural wall upon which the insulating layer was to "ride" into position in the building frame. Most applications of this type wall anticipated that the insulating layer was to be plastered over after erection of the wall panel. Design of the open-face wall panel used by Insulation Industries, Vancouver, B. C., Canada, is shown in Fig. 3. Layer A was a 4-in. hard, dense layer of regular concrete reinforced as required by the designer. Layer B, also reinforced, was the insulation layer of vermiculite concrete. After the wall panels were erected, with layer A exposed to the weather and layer B inside, the joints between panels were poured. The same general type of sandwich wall construction was used on the Owen Building, Columbia, S. C. The wall panels for this building were cast on the concrete floors and tilted into place. Layer A in this case was 2 in. of regular concrete and layer B was 6 in. of vermiculite concrete.

Mineralized wood chip sandwich panel

This sandwich wall design was developed in the United States and reported by P. M. Grennan.* It was designed to be used as cladding for the steel frame of an industrial building. However, it is reported here because it seems to be an excellent design and one that could well be used in site-cast tilt-up construction.

The design of the 8 x 8-ft panels is shown in Fig. 4. Each panel was composed of two outer walls of regular concrete $1\frac{3}{4}$ in. thick reinforced with 4 x 4–10/10 mesh. Lightweight precast insulation was $1\frac{1}{2}$ in. thick using chemically mineralized wood chips as aggregate. The two outer shells were

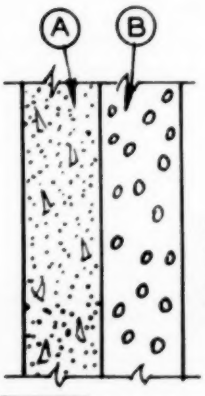

Fig. 3—"Open-face" sandwich panel: (A) hard, dense layer of regular concrete and (B) insulation layer of lightweight concrete

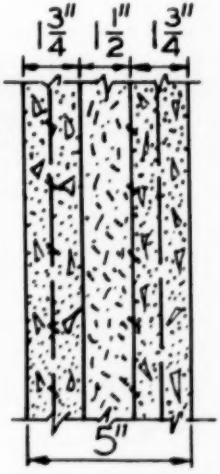

Fig. 4—Sandwich panel using chemically mineralized wood chips as the aggregate in the insulation filler

*Grennan, P. M., "Precast Sandwich Wall Makes Costs Tumble," *Engineering News-Record*, Jan. 24, 1952, pp. 34-36, 39.

not tied through the insulation by shear ties as is customary with other sandwich panels—there were shear ties only at the periphery of the panel. The type of sandwich material was such that it could be used to transfer horizontal shear from one face to the other through bonding of the two outer faces to the insulation.

Edges of the panel were tongue-and-groove joints. Panels were assembled with a 1 in. wide impervious cellular rubber strip inserted at the center of all joints. This rubber strip was secured in place during erection by an adhesive. Joints were completed by packing with oakum and caulking.

Cellular glass sandwich panels

A design similar to the above panel, but differing enough to be reported is the sandwich panels used by Celanese Corp. of America on its pulp mill for Columbia Cellulose Co., Ltd., Watson Island, B. C., Canada, reported by S. B. Roberts.* The panels (Fig. 5) were 6 x 10 ft and consisted of a 2-in. layer of cellular glass insulation and two wire mesh reinforced slabs of 3000-psi concrete, tied together with channel-shaped strips of expanded metal. The inner face was 1½ in. thick and incorporated threaded metal inserts near the corners for fastening to the building frame.

An outstanding sandwich panel design is the one designed by the architectural firm of Shaw, Metz, and Dolio, of Chicago, for the Great Lakes Naval Training Station housing project constructed in 1951 by Corbetta Construction Co., Inc., New York, and Price Brothers, Dayton, Ohio. The design is shown in Fig. 6.

These walls were cast on plastic-faced plywood. All door and window frames and forms for other openings were set into the forms prior to concreting. The 4 x 4–4/4 reinforcing mesh was then placed on chairs and concrete placed to the approximate half thickness of the walls. The transit-mixed concrete had a minimum 28-day strength of 4000 psi and was made with 6¾ bags of high early strength cement per cu yd and pea gravel aggregate. When the concrete had been vibrated and screeded, 1½ in. thick cellular glass insulation sheets (giving a U value of 0.199) were set on the concrete. Shear ties consisting of 4 in. wide strips of 4 x 4–6/6 mesh were inserted between transverse rows of Foamglas blocks and embedded about

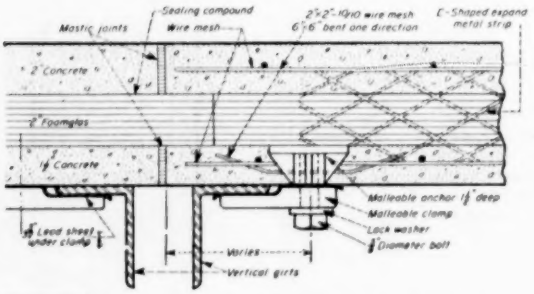

Fig. 5—Sandwich panel using cellular glass insulation as the filler material

*Roberts, S. B., "Sandwich Walls Precast for Pulp Mill," *Engineering News-Record*, Feb. 22, 1951, pp. 32-34.

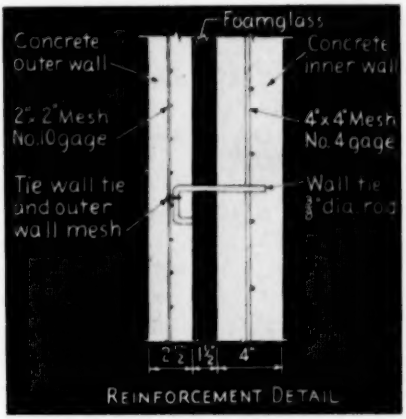

Fig. 6—Another sandwich-type precast concrete wall using cellular glass insulation. Inner and outer concrete walls were tied together with J-shaped metal wall ties

1 in. in the concrete. J-shaped, ⅜-in. round bars were then punched through the insulation and into the lower layer of concrete to tie the inner and outer layers together. Precut sheets of 2 x 2–10/10 wire mesh were placed on top of the insulation blocks and the rest of the concrete placed. The shear ties were wired to the 2 x 2-in. mesh. The top layer of concrete was struck off with a screed and the surface steel troweled and broomed. A decorative pattern was impressed into some parts of the wall before the concrete set.

Foam concrete sandwich panels

Foam concrete has been used as a filler for sandwich panels by Lakeland Engineering Associates, Inc., of Florida (Fig. 7). The filler used was made with a preformed foam using Mearlcrete P foaming agent and had a density of 25 to 28 lb per cu ft, dry.

Bonding properties of foam concrete are considered good but it is recom-

Fig. 7—Tilt-up sandwich wall using foam concrete filler. This is a 12-in. wall using 8 in. of foam concrete as the insulating sandwich material with high strength dense reinforced concrete on each face

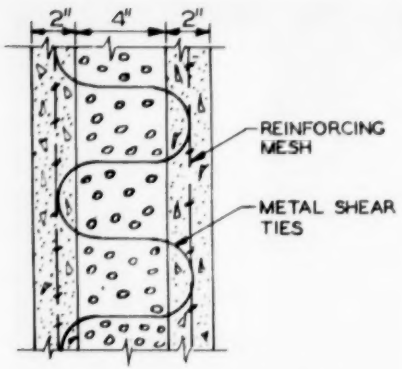

Fig. 8—Swedish sandwich wall panel using lightweight expanded shale concrete as the insulating filler

mended that steel ties be used between filler and outer concrete shells as in the preceding types of panels.

Expanded shale sandwich panel

One of the few true tilt-up sandwich wall panels is that recently developed in Sweden and reported to the author by Hilger Rohfors of Svenska Cementforeningen. This panel (Fig. 8) was composed of 2-in. outer shells of regular dense concrete with 4 in. of expanded shale concrete between. The panel had metal ties between the two outer layers of dense concrete.

The Swedish builders believe that the moisture causing difficulty, as far as diminishing the insulation capacity of the concrete is concerned, is the water incorporated in the structure during construction. It is therefore important to use dry, lightweight concrete with wide communicating pores that dry out rapidly. An expanded shale aggregate, ¼ to ⅝ in. with no fines, was chosen—giving a porous concrete with a volume weight of 50 lb per cu ft. The U value of such a sandwich wall is 1.6 Btu per hr per sq ft per deg F.

In casting the panels, the shale concrete is placed on the fresh lower 2 in. of regular concrete. It has a rather dry consistency and the top layer of regular concrete can be placed immediately. The speed in casting is thus better than a foamed or slow setting insulating concrete. The shale concrete is also considered to be more economical than foam concrete, cinders, or other lightweight concrete mixes. The open textured shale concrete also offers good bond between the concrete layers.

It is this type of sandwich panel design that the author believes will give the best results and greatest economy in site-cast, tilt-up concrete construction. This type of panel, cast in tilt-up sizes, would cost one-half to two-thirds that of smaller sandwich panels with expensive insulative material.

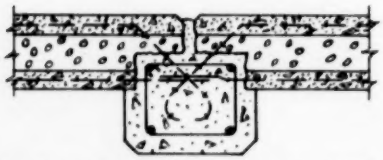

Fig. 9—Wall column design for Swedish sandwich wall

The Swedish builders have found that for buildings with ordinary inside humidity, there seems to be no need for a vapor barrier, provided good dense concrete is used in the inside and outside layers. Bond between the layers in the panel seems to be sufficient for lifting operations, particularly with the two outer faces connected with shear ties. One method of connecting the two faces to the cast-in-place vertical element between panels is shown in Fig. 9.

Vermiculite sandwich panels

For several years the author has been testing various sandwich panels with a vermiculite concrete filler material. It is placed in a plastic state on the fresh lower layer of regular concrete and a 2-in. top layer of regular concrete placed on the vermiculite layer as soon as it has set sufficiently to carry the top layer. Both reinforced and unreinforced outer shells were tested.

From these tests the design in Fig. 10 has evolved. The two outer shells are 1¾ in. thick regular concrete reinforced with 2 x 2–12/12 wire fabric. The sandwich material is 2½ in. of vermiculite concrete, approximately a 1:8 mix. The two shells are attached by ¼-in. sinusoidal shear reinforcing spaced vertically with a maximum 4-ft center-to-center spacing. Possible connection details to other structural elements are illustrated in Fig. 11a, b, and c.

DESIGN FUNDAMENTALS FOR CONVENTIONAL SANDWICH WALLS

Sandwich materials

Ideal sandwich materials are those with low density, relatively high compressive strength, high shear strength, good bonding characteristics, high insulative qualities, and low cost. Such an ideal material is hard to find, for those that meet some of the requirements do not always meet with others. Available materials can be divided into about four divisions: (1) cellular glass materials and plastic foam; (2) compressed and treated wood fibers in cement; (3) foam concrete; and (4) lightweight concrete. Table 1 gives design information on these materials. Also, excellent design data on lightweight aggregate concrete which can be used in sandwich construction is given in the ACI JOURNAL paper "Lightweight-Aggregate Concrete."*

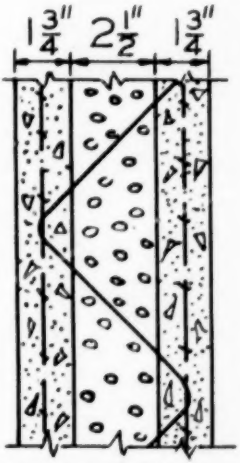

Fig. 10–Sandwich panel using vermiculite concrete as filler material

For the factory cast, precision-made type of sandwich wall panel, such materials as cellular glass or compressed wood fibers might be used. But for the job-cast type of large tilt-up sandwich wall panel, the author believes that the most economical and satisfactory filler material is the lightweight concrete mixes

*Kluge, Ralph W., Sparks, Morris M., and Tuma, Edward C., "Lightweight-Aggregate Concrete," ACI JOURNAL, May 1949, *Proc.* V. 45, pp. 625-644.

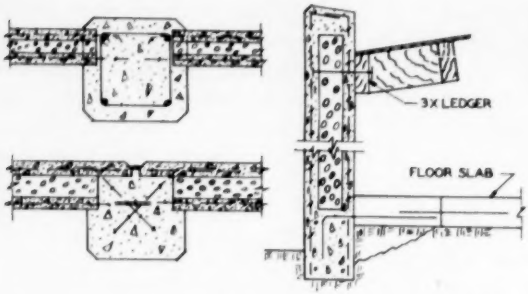

Fig. 11a (top left)—Column detail for sandwich panel construction. Fig. 11b (bottom left)—Alternate column detail. This joint can be either formed and cast in place or shotcreted. Fig. 11c (right)—Detail of parapet and tie to floor slab

given in Table 1. From the data in Table 1, the most promising material seems to be perlite concrete. Another possibility is the use of machine processed expanded slag. At a mix of approximately 1:9, a good weight can be obtained, with acceptable strengths, medium water adsorption, and a good K factor.

Design of the sandwich

Heat transmission is an important factor of sandwich panel walls. By the laws of thermodynamics, the total resistance to the heat flow through the wall shown in Fig. 12 is equal numerically to the sum of the resistances in series. The resistances are the outer surface film of the wall, the material in the wall itself, and the surface on the other side of the wall.

The drop in thermal gradient can be represented by the wavy line in Fig. 12. At the first resistance, the film f_i, there is a drop in the gradient necessary to overcome the resistance of the film. The same is true for the material of the wall and the surface film f_o on the other side. The sum of these resistances is the total resistance. The over-all coefficient of heat transmission U is the

TABLE 1—DATA ON SANDWICH MATERIALS

Material		Physical state	Density, lb per cu ft	Bonding ability	Compressive strength, psi	Thermal conductivity K, Btu per sq ft per hr per in. per deg F	Water adsorption by volume, percent	Cost	Remarks
Cellular glass		rigid	9 to 11	good	125–140	0.38	0.20	medium high	
Expanded polystyrene		rigid	1.3 to 2	probably fair	10–35	0.23–0.30	5–6	high	more research needed, strength low
Compressed wood fibers in cement		rigid	32–36	very good	200	0.43	fair	medium	shear ties might be omitted
Foam concrete		plastic	38–60	very good	300–500	1.2–1.7 (estimated)	fair	low to medium	difficult to control
Vermiculite concrete	Mix 1:8	plastic	25–32	fair	100–200	0.90–1.10	44	medium	holds water
Perlite concrete	Mix 1:8	plastic	23–28	fair	100–200	0.80–1.00	25	medium	
Pumice concrete	Mix 1:8	plastic	57–62	good	960–1270	1.70–1.90	28	medium low	
Expanded slag concrete	Mix 1:9	plastic	60–62	very good	290–580	1.55	20	medium low	a machine processed expanded slag

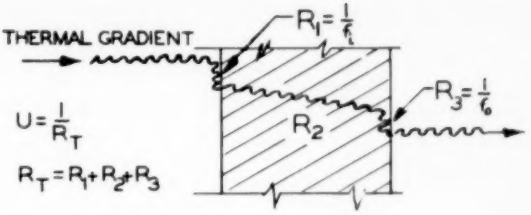

Fig. 12—Heat flow through wall section

reciprocal of the sum of the resistances.

$$U = 1/R_T \quad (1)$$

$$1/R_T = 1/(R_1 + R_2 + R_3)$$

$$R_1 = 1/f_i \; ; \; R_2 = x_2/K_2 \; ; \; R_3 = 1/f_o$$

$$U = \frac{1}{1/f_i + x_2/K_2 + 1/f_o} \quad (2)$$

Now we are ready to design a sandwich wall from the insulation standpoint (Fig. 13). Let us assume we are using an expanded shale concrete with a conductivity K of 1.5 at a weight of 50 lb per cu ft and we want to obtain a sandwich panel with a U of 0.27 Btu per hr per sq ft per deg F. For regular concrete, $K = 12$; f_i in this case is 1.65 and f_o is 6.0. A 2-in. thickness is chosen for the outer shells. By trial-and-error methods, the filler thickness could be increased or decreased until the desired U was achieved. Better yet, by setting the resistance of the sandwich at x, and retaining the other dimensions and factors, we can solve for the thickness that would give the desired U.

$$U = 0.27 = \frac{1}{0.605 + 0.167 + R_2 + 0.167 + 0.167}$$

$$0.27 = \frac{1}{1.10 + R_2}$$

$$R_2 = 2.6$$

$$\text{But } R_2 = x_2/K_2 = x_2/1.5 = 2.6$$

$$x_2 = 2.6 \times 1.5 = 3.9 \text{ in.—say 4 in.}$$

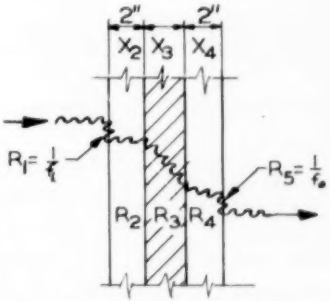

Fig. 13—Sandwich wall design example with 2-in. outer shells

Outer shell design

Outer shells of a sandwich panel should be of dense high strength concrete of at least 2500 psi strength. Preferred practice would be to use 3000-psi concrete, with a 6-sack mix, designed to give maximum density, compressive strength, and watertightness. It has been found in practice that this concrete is best placed with a slump of about 4 in., ½ to ⅜ in. maximum aggregate, and an admixture to aid workability. The shells are so thin that internal vibration is out of the question and screed vibration is recommended.

Tests on sandwich panels with unreinforced outer shells have shown that such panels perform surprisingly well. Most designers believe that a minimum mesh should be used in the outer shells. However, with the right sandwich material, the outer shells could be unreinforced, especially if the panels

TABLE 2—RECOMMENDED DESIGN DIMENSIONS AND SPECIFICATIONS FOR OUTER SHELLS

Type of panel	Minimum shell thickness, in.	Minimum strength, psi	Maximum aggregate, in.	Admixture		Reinforcing	
				Air-entraining	Early strength	Shell reinforcing	Shear ties
Conventional reinforcing, rigid filler	1¼ (factory conditions)	2500	⅜	yes	yes	mesh	yes in most cases
Conventional reinforcing, plastic filler	1½	2500	½	yes	yes	mesh	yes
Prestressed, plastic filler	1½	3000	½	yes	yes	could be eliminated	yes

were prestressed. If more steel is needed to resist bending stresses, then that reinforcement would also have to be placed in the outer shells. For a sandwich material such as compressed wood fibers in cement, placed in staggered sheets, there would be less need for reinforcing in the outer shells; however, this requires more research and investigation.

As regards minimum thickness, this depends upon the type of aggregate available, location of the structure, weather conditions, the "know-how" of the organization fabricating the panels, and other variables. Table 2 gives recommended dimensions and specifications for various type panels. It has been proved that a minimum thickness of 1¼ in. can be obtained under factory conditions. Casting at the job site, under less satisfactory conditions, upon a surface considerably different from those usually used for plant produced channel slabs and thin shells, a minimum thickness of 1½ or 1¾ in. is recommended. Where labor and technical know-how leaves something to be desired, a 2-in. thickness is recommended.

Shear ties

Use of shear ties, to connect one shell to the other and to transfer shear from the center of the panel area to the boundary connections, is recommended for the majority of sandwich panels regardless of filler material. The only material that appears to be sufficiently shear resistant to warrant omission of ties is the compressed treated wood fibers in cement. Even then, most designers prefer to use minimum shear ties in tilt-up construction where the sandwich panels might be as large as 33 ft high and 20 ft wide. For smaller panels using a wood fiber filler and fabricated in a plant, where the panels

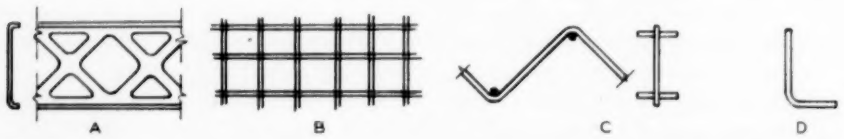

Fig. 14—Types of shear ties: (A) thin-gage expanded metal, (B) strip of electrically welded mesh, (C) specially designed bent wire, and (D) specially designed J-bar

are intended as cladding for a steel frame, shear ties might well be omitted. Some typical shear ties are shown in Fig. 14.

EXAMPLE WALL PANEL DESIGN

It is desired to design a wall panel for a warehouse, Type I construction, where the height of the wall from floor diaphragm to roof diaphragm is 25 ft. The wind design load is 20 lb per sq ft, and the seismic coefficient is 20 percent of gravity. The desired U for the wall panel is 0.22 Btu per hr per sq ft per deg F. A comparison between a brick, concrete, and sandwich wall gives the following:

Construction features	Common brick	Regular concrete	Sandwich panel using vermiculite filler
U, Btu per hr per sq ft per deg F	0.22	0.22	0.22
Wall thickness, in.	15	35	6
Weight of wall per sq ft, lb	140	435	56
Cross-sectional area of wall per 4-ft width, sq in.	720	1680	192

From this it is evident that a sandwich wall will give the desired insulation at a reasonable cost.

It is decided (because of easy availability) that a machine processed expanded slag could be used as lightweight aggregate for the sandwich filler. Such a filler has the following properties: (1) weight—65 lb per cu ft; (2) compressive strength—560 psi; (3) transverse strength—148 psi; (4) thermal conductivity K—1.60 Btu per hr per sq ft per deg F per in.; and (5) good workability if air entrainment is used.

Sandwich thickness

We now calculate the thickness of insulation required to obtain a U of 0.22. It is believed that 1¾-in. outer shells of regular concrete will be satisfactory. For the concrete shell, $x_1 = 1.75$ in., $K_1 = 12$; for the expanded slag filler, x_2 thick, $K_2 = 1.60$; inside face $f_i = 1.65$; outside face $f_o = 6.0$.

$$U = 0.22 = \frac{1}{1/f_i + x_1/K_1 + x_2/K_2 + x_3/K_3 + 1/f_o}$$

$$= \frac{1}{1/1.65 + 1.75/12 + x_2/K_2 + 1.75/12 + 1/6}$$

$$R_2 = x_2/K_2 = x_2/1.6 = \frac{1 - 0.235}{0.22} = 3.48$$

$$x_2 = 5\tfrac{1}{2} \text{ in. thick}$$

This is enough filler material to call for a check calculation using a more expensive material, for example, cellular glass.

$K_2 = 0.38$

$R_2 = x_2/K_2 = 3.48 = x_2/0.38$

Therefore $x_2 = 0.38 \times 3.48 = 1.32$—say $1\tfrac{1}{2}$ in.

We would like to stay with a lightweight concrete filler if possible, so we calculate the thickness required using perlite concrete.

$K_2 = 0.85$
$x_2/0.85 = 3.48$
$x_2 = 0.85 \times 3.48 = 2.93$—say 3 in.

We now know the required thickness for three filler materials. It is now necessary to investigate the required section to span 25 ft from floor to roof or the 20-ft distance from column to column. The bending moment from both seismic origin and wind origin must be checked to see which condition is critical in design.

Wind bending moment—Span = 25 ft; $W = 20 \times 25 = 500$ lb
$M_{max} = WL/8 = 500 \times 25 \times 1.5 = 18{,}800$ in.-lb

Seismic bending moment—Check weight of wall: (1) 3½-in. regular concrete = 42 lb, (2) 3-in. perlite concrete = 7 lb, (3) total = 49 lb, use 50 lb. The unit lateral force equals $0.20 \times 50 = 10$ lb per sq ft; therefore, the wind force governs design.

Check section using 3-in. perlite filler (Fig. 15)—

$$S = \frac{b_1 h_1}{6} - \frac{b_2 h_2^2}{6} = 66 \text{ cu in.}$$

$F = 18{,}800/5 = 3750$ lb per ft of width
Required steel area $A_s = 3750/30{,}000 = 0.125$ sq in., which is high for mesh

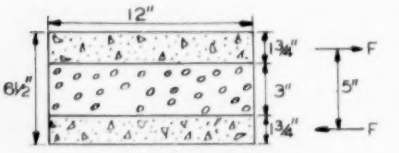

Fig. 15—Design example

Checking concrete stresses:
$f_c = 3750/(12 \times 1.75) = 180$ psi
ignoring steel:
$f_T = f_c = M/S = 18{,}800/66 = 275$ psi

The concrete stresses are satisfactory, but the steel requirements are too high for economical design.

Now let us check the bending moment when the wall panel frames from column to column, with the columns 20 ft center-to-center. Since there will be a poured joint at the columns, it is safe to assume that the moment will be at least $WL/10$, and that the span of the wall itself will be only 19 ft. The new bending moment will be:

$M = WL/10$ $W = 19 \times 20 = 380$ lb
$M = 380 \times 19 \times 1.2 = 8690$ in.-lb
$F = 8690/5 = 1720$ lb
$A_s = 1720/30{,}000 = 0.058$ sq in.

A sandwich material 4 in. thick would allow the use of 2 x 2–12/12 mesh, which is a well-balanced mesh for this thickness.

Checking for K factor required for a 4-in. filler: $R_2 = 4/K = 3.48$ and $K = 1.15$.

The final decision for this situation is to frame the building columns (in this case we are assuming the spacing can be adjusted) on an 18 ft 6 in. spacing (span of wall 17 ft 6 in.). Design bending moments will then be adjusted as follows:

$W = 17.5 \times 20 = 350$ lb
$M = WL/10 = 350 \times 17.5 \times 1.2 = 7400$ in.-lb
$F = 7400/5 = 1480$ lb
$A_s = 1480/30{,}000 = 0.0495$ sq in.

The 2x2–12/12 mesh provides an A_s of 0.052, which is considered satisfactory.

Shear ties

The shear ties will be designed for the shear developed by the 18 ft 6 in. column spacing, but there must be sufficient ties to erect the panel. If the filler material is fairly shear resistant, it will take all the shear during erection. For this example the shear ties will be horizontal with 3-ft center-to-center spacing. Shear ties in the region of the embedded pick-up units will run vertically to take care of the shear from lifting.

Maximum shear $= 350/2 \times 3 = 520$ lb per tie at 3-ft spacing. Assume all shear in 45 deg diagonal tie (critical).

$P = 1.41 \times 520 = 735$ lb
$A_s = 735/1500 = 0.049$ sq in.

Use $\frac{1}{4}$-in. ϕ wire at 3-ft center-to-center; 45 deg bend at critical ends, 30 deg at center past quarter-point.

For the erection loads use $\frac{1}{3}$-in. ϕ wire at 2 ft 10 in. centers; four rows 6 ft long at pick-up points.

PRESTRESSED SANDWICH PANELS

The author believes that the ideal precast sandwich wall panel design is one that incorporates prestressing in the outer shells. Although it is not economically feasible for the smaller 8 x 8-ft or 10 x 10-ft panels, it could be used advantageously for the large two- and three-story tilt-up panels that are possible today. Prestressing of sandwich panels has already been tried on an experimental basis (Fig. 16). The following example is worked out for a three-story prestressed sandwich wall panel.

Fig. 16—Construction of prestressed sandwich wall panel. Left—Forming the panel with diagonal prestressing units in place. The first 2 in. of regular concrete has been placed. Filler material was 4 in. of vermiculite concrete, which was covered with another 2 in. of regular concrete. Right—Testing wall panel to determine buckling of shells and shear resistance of panel. Under heavy localized shear stresses, the shells and sandwich separated at the lower right corner of the panel

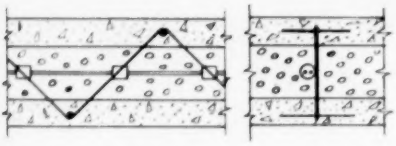

Fig. 17—Shear tie design with two-wire prestressing unit centered in panel

Let us use the general wall design developed on previous pages, using 1¾-in. outer shells and a 3-in. sandwich of perlite or expanded slag concrete. The forces that will control the design will be the wind loads (20 lb per sq ft) normal to the wall since the walls are non-load-bearing. Distance between floors is 11 ft 4 in., but the design will be made for 12 ft.

$W = 12 \times 20 = 240$ lb

$M = WL/8 = 240 \times 12 \times 1.5 = 4320$ in.-lb

Since the wall panel has a section modulus of 66 cu in. for every foot of width, and since the design moment is 4320 in.-lb per foot of width at the critical section, tensile stresses from bending would be about 66 psi. If it is desired to have no tension in the concrete at design load, the outer shells can be prestressed, building in enough compression in the outer shells to overcome this 66 psi tension. Since there is 42 sq in. of shell material per foot of width, approximately 2772 lb per ft of prestress force is needed; this is 8316 lb every 3 ft. This can be accomplished by two 0.196-in. diameter prestressing wires spaced at 3-ft centers and stressed initially to 10,000 lb.

It is proposed that the shear ties also be used to center the prestressing wire as shown in Fig. 17. Since the prestressing wire is to be in the center of the light density sandwich material, a coating of gilsonite should be all that is needed to keep the wire from bonding to the sandwich material.

Pick-up analysis

It is proposed that this wall be picked up by four-point pick-up as shown

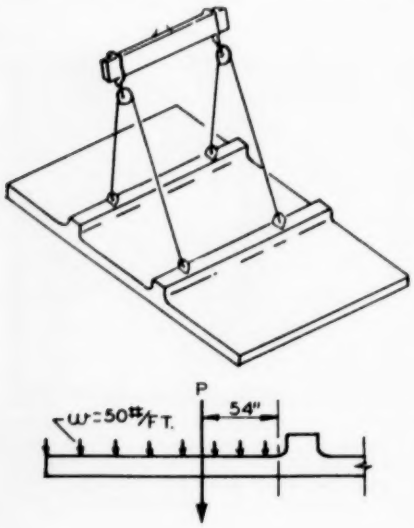

Fig. 18 (top)—Four-point pick-up proposed for tilt-up erection of three-story prestressed sandwich wall. Fig. 19 (bottom)—Critical section of wall panel in figuring erection stresses

in Fig. 18. The critical section would be that area just to the top of the top bond beam and the top pick-up points. For a solid panel the bending moment would be calculated as follows (Fig. 19).

$P = 9 \times 50 = 450$ lb
$M = 450 \times 54 = 24{,}300$ in.-lb
$f_T = M/S = 24{,}300/66 = 358$ psi

The prestressing units will be 3 ft on center, with double units at the second wire in from the edge, giving a total of 11 two-wire units. The calculations will be based on the assumption that only ten are acting.

$$f_c = \frac{10{,}000 \times 10}{18.8 \times 42} = 126 \text{ psi}$$

The tensile force left is then $358 - 126 = 232$ psi. This force can be resisted by reinforcing steel, size of which is

$A_s = (232 \times 42)/30{,}000 = 0.32$ sq in. per ft

Use $\frac{3}{8}$-in. ϕ bars on 8-in. centers 8 ft long centered on the bond beam alternated with $\frac{3}{8}$-in. ϕ bars on 8-in. centers 5 ft long.

It has been the author's experience that it is possible to rely on the modulus of rupture of the concrete during erection. A calculated tensile stress of 232 psi is not excessive, and with a test cylinder ultimate compressive strength of 2000 psi, it would be possible to erect the panel without the extra steel just calculated.

The total weight of this panel would be: (1) sandwich area—560×50 in. = 28,000 lb; (2) beams—150×37 ft = 5550 lb; (3) solid area—80×37 ft = 2960 lb; giving a total of 36,510 lb or say $18\frac{1}{4}$ tons. This could be handled with a 25-ton crane, although a 35-ton crane would probably be preferable.

Cost of fabrication

The cost per sq ft of wall of fabricating the panel outlined above would be:

1.	Preparing casting platform		0.01
2.	Forming panels		0.12
3.	Reinforcing steel		
	Bond beams	0.250 lb @ 11	
	Periphery	0.300 lb @ 11	
	Shear ties	0.150 lb @ 12	
		0.700 lb @ 12 = $83	0.13
4.	Placing concrete		
	Outer shells		0.20
	Filler material		0.25
	Finishing one side		0.06
5.	Prestressing		0.13
6.	Erection		0.04
7.	Cast-in-place column		0.16
8.	Sacking, patching, etc		0.04
	Total (per sq ft)		$1.25

The estimated construction cost of $1.25–1.35 per sq ft of erected panel includes the cost of the cast-in-place columns between panels. Some of the

smaller factory cast sandwich wall panels have cost as much as $2.00 per sq ft, not including the building frame. When the horizontal steel framing to support the panels is included, plus vertical column costs, the cost per sq ft of building exterior shell approaches $3.00 per sq ft of wall, column, and wall framing.

CONCLUSIONS

There is no doubt that the increasing use of precast concrete in the United States will also offer opportunities to utilize tilt-up sandwich wall construction.

Building design and construction is being regulated more carefully. The pattern is clearly set that code requirements for exterior walls for Type I, II, and III buildings will call for a 4-hr fire rating. It is impossible to obtain a 4-hr rating with thin-shell construction of 1¼ to 2 in. thick. Some building codes, following the example of the City of Los Angeles, require an 8-in. thickness of regular concrete for a 4-hr fire rating. Large tilt-up walls 8 in. thick are difficult to erect because of the weight. The proposed sandwich wall panel—33 ft high, 6½ in. thick, with two outer shells 1¾ in. thick, the filler of lightweight concrete, and prestressed—would meet the reduced weight requirements, provide excellent insulation, and offer one of the best opportunities to date to prestress wall panels economically.

Production of Sand and Gravel*

By STANTON WALKER†

SYNOPSIS

Production of sand and gravel is discussed under three main groupings: (1) development of deposits, (2) transportation to the plant, and (3) processing. Briefly detailed are prospecting and exploration, excavation, and transporting methods. Described in more detail are processes for washing, classifying, removing deleterious impurities, and crushing oversized materials. A list of 32 selected references completes the report.

INTRODUCTION

Sand and gravel are consolidated granular materials resulting from the natural disintegration of rocks. They generally occur together, variably proportioned in widely available deposits. Their production consists of extraction, transportation to the plant, and processing—the latter generally including washing, screening and otherwise classifying to size, crushing of oversize, and the elimination of deleterious impurities to the extent practicable.

Sand and gravel occur in a wide variety of deposits both above and below water. These deposits may consist of banks, pits, or the beds of streams, lakes or other bodies of water. The banks and pits may extend below the water level. Sand and gravel originate from the weathering of rock and its transportation by water. It may come from material deposited by glaciation or from residual materials broken down in place or from a combination of these. Consequently, mineralogically they are whatever their parent rocks were. The type of the deposit and the nature of the material in it dictates an extremely wide range in methods of production from extraction through final processing.

A first step toward the production of sand and gravel is a detailed exploration of the selected deposit to determine characteristics and quantities of material. The sand and gravel should be examined as to mineral composition, quantity and nature of impurities, soundness, strength, and size and grading. The question of whether or not the material can be economically produced to meet the requirements of applicable specifications is one that can be answered only by sound engineering judgment, based on experience coupled with a careful survey of the deposit and the need for its products.

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This short paper on sand and gravel production permits no more than a brief discussion of fundamental principles. A comprehensive treatment of the subject is impractical except in a work very broad in scope. The literature, while considerable, has not been well correlated. It is hoped, however, that this statement of fundamentals together with references to sources of information will serve a useful purpose.

PROSPECTING AND EXPLORATION

Sand and gravel deposits, by the nature of their occurrence, are evaluated as to quantity and quality only by painstaking exploration. The exploration falls in two parts—a preliminary one to estimate the extent and general nature of the material and, if that is promising, a detailed one which will not only furnish reasonably accurate information as to quantities available, but also as to the quality of the material.

There are a variety of approaches to preliminary exploration. Information available from state topographic, hydrologic, and geological surveys, and from other federal and state government agencies often is of considerable value. Outcroppings, visible deposits under water, and miscellaneous available excavations should be examined. Recent developments in geophysical methods offer promise for rapid preliminary surveys. Resistivity methods, which indicate changes in composition of underground strata by differences in electrical resistance, seem most applicable.

Before any final answer can be secured, exploration methods must be used which provide representative samples. The relative amounts of sand and gravel must be determined and evaluated in the light of market demand and specification requirements. Grading of the material must be examined to determine if specification requirements can be met economically. The nature and amount of deleterious materials and the feasibility of their economical removal must be studied. The problem is complicated by the inherent lack of uniformity of many sand and gravel deposits due to the natural processes by which they were formed.

Exploration methods which permit securing representative samples from land deposits include test pits and trenches, driving of pipes and excavation of material from the pipes by special bucket or other means, earth augers, churn drills, and other procedures for getting a cross section of the material. For underwater deposits, dredges, both pump and ladder, cranes and buckets, draglines, and similar straightforward devices are used. There are no short-cut methods.

There is relatively little literature dealing directly with prospecting and exploration for sand and gravel. In 1932, J. R. Thoenen of the U. S. Bureau of Mines prepared a 50-page discussion of the problems as a whole.[1] He discussed, in somewhat general terms, the occurrence of sand and gravel and the various methods of exploration applicable to different types of

deposits. The paper is out of print but should be available in many technical libraries.

A symposium on prospecting was held at the 1950 annual meeting of the National Sand and Gravel Assn. R. Woodward Moore of the Bureau of Public Roads discussed geophysical methods of subsurface exploration and gave a selected list of references to the literature.[2] Experiences of members of the industry with augers, adaptations of well-drilling equipment, resistivity methods, and in underwater exploration are described in references 3, 4, and 5.

General information on prospecting may be found in the Bureau of Reclamation *Concrete Manual*, in the American Institute of Mining Engineers publication, *Industrial Minerals and Rocks*, particularly the chapter on "Sand and Gravel," by Bror Nordberg,[6] and in the American Society for Testing Materials 1948 *Symposium on Mineral Aggregates*, particularly the chapters on "Distribution of Mineral Aggregates," by K. B. Woods,[7] and "Production and Manufacture of Fine and Coarse Aggregates," by N. C. Rockwood.[8]

DEVELOPMENT OF DEPOSITS

Excavation of material from the deposit is the first step in production. For bank and pit deposits above the water level, excavation is ordinarily by power shovel, dragline, drag scraper, or slackline cableway. Earth-moving, tractor-combination machines sometimes are used in shallow deposits. Various methods, including light blasting, are used to break down the face of the pit or bank to make the material more conveniently available for excavation.

Most land deposits are covered by overburden. If it is light and sandy, it may be excavated along with the sand and gravel and removed in the processing plant. If it is heavy, it should be stripped ahead of the excavating operation. The same, or similar, equipment may be used for stripping as for excavation.

Draglines and cableways may also be used to excavate below the water level in wet pits and shallow streams. However, if the nature of the material and the quantity of water permits, it is common practice to use suction dredge pumps. After a pit is opened, the dredge may be used to excavate the above-water material as well, it being broken down into the water by under-cutting, by light blasting, or by other means.

For marine deposits both suction and ladder dredges are used. The suction dredge generally delivers the material directly to the processing plant on land, or it may pump to a sump from which the material is pumped again. Less frequently the material may be pumped to barges in which it is transported to the processing plant.

The ladder dredge is used for deposits which are too consolidated or too coarse for suction dredges and for other reasons which may be dictated by economy of operation. Sometimes ladder dredges deliver the materials

direct to barges for transportation to processing plants but, more frequently, except in large wet-pit operations, the processing plant is on the same hull as the dredge.

Excavation is also done with crane and bucket, with dipper dredges and, infrequently, by hydraulic sluicing. Excavation methods and excavating equipment used for production of sand and gravel are those common in the mining field, and are extensively described in the literature. The magazines *Pit and Quarry*[9] and *Rock Products*[10] pay particular attention to sand and gravel production and almost any issue will include descriptions of one or more operations. Another Bureau of Mines publication by Thoenen[11] on developing sand and gravel deposits, written in 1933, deals with excavation from different types of deposits and gives references to descriptions of a number of specific operations.

The *Pit and Quarry Handbook*[12] includes comprehensive chapters on dredging, stripping, pumping, and excavating pertinent to any consideration of recovery of materials from selected sources. Information of a more general nature will be found in the papers by Nordberg[6] and Rockwood[8] previously cited. There is also much general information, particularly on dredges and pumps, in Taggart's *Handbook of Mineral Dressing*.[13]

TRANSPORTATION TO PLANT

After excavation the material must be taken to the processing plant. Some of the methods of excavation incorporate methods of transportation to the plant—as the suction dredge, the earth mover and, sometimes, the cableway scrapers. For underwater deposits, barges may be used to transport unprocessed material from dredge to plant although more often processing is done on the dredge. When shovels, draglines, or similar equipment are used, independent means must be provided. These include narrow and standard gauge railroads, trucks (of all kinds and up to about 20 cu yd capacity), and belt conveyors. Each method has its place and its economic advantages, depending upon the nature of the deposit.

Transportation from point of excavation to the processing plant is sometimes done in two stages—a "surge pile" being introduced between the two. In the case of a dredge pump operation, material may be pumped to a sump and picked up by a booster pump for transportation to the plant. In the case of a land operation, excavated material may be transported to a storage pile (surge pile) by a selected means and then conveyed from that pile by any other selected means, generally a belt. This intermediate storage serves, among other things, to facilitate maintaining a uniform feed to the plant by divorcing it from complete dependence on the rate of excavation.

The chapter on "Intraplant Transportation" in the *Pit and Quarry Handbook*[12] treats the several methods in considerable detail. Taggart[13] also deals with transportation, including some information on costs, and there is general information in the previously cited works of Nordberg[6] and Rockwood.[8]

PLANT PROCESSING

Plant processing includes washing, screening and otherwise classifiying to size, crushing of oversize, and the reduction of deleterious impurities to the extent practical. Impurities which are soluble or suspendable in water generally can be washed out satisfactorily. Sizing and classifying can be done with considerable accuracy within the limits of the sizes available, although costs are greatly increased if it is necessary to waste much material or if too many or too close separations, particularly in the sand sizes, are required. Deleterious materials may be reduced, but there is a limit beyond which such operations are neither feasible nor economical.

The processing plant essentially is a simple combination of conveyors, screens, crushers, washing and classifying equipment, and storage and loading facilities. Any attempt to illustrate a typical one or to show a typical flow sheet would be misleading. The plant must be tailored to suit the deposit and the market. In the simple case of a land deposit, a plant might consist of the following elements:

1. A hopper, or equivalent, to receive material transported from the deposit or surge pile. Generally this hopper will be covered with a "grizzly" of parallel bars, or equivalent, to screen out boulders too large to be handled by the plant—in most cases a relatively small proportion of casual material.

2. From this receiving hopper the material will pass through a gate, controlled by a "feeder," of which there are numerous designs, to a conveyor on which it is carried to the top of the plant on to the scalping screen. A belt conveyor is generally the most economical. Bucket elevators have been used and are still used but less than in the past. Skip hoists have been used, but sparingly.

3. The scalping screen separates oversize material from the smaller, marketable sizes.

4. Material passing the scalping screen is fed into a battery of screens, either vibrating or revolving, the number, size, and arrangement of which will depend on the sizes and number of sizes to be made. Water from sprays is introduced from the beginning and is applied throughout the screening operation. From these screens the different sizes of gravel will be discharged into bins or on to conveyors for transportation to stockpiles or, in some cases, to crushers and other screens for further processing. The sand passes to classifying and dewatering equipment and from there to bins or stockpiles.

5. The oversize from the scalping screen passes to the crushers—the size, number, and arrangement of which depends upon the quantity and nature of the material to be crushed.

6. Crushed material may be re-elevated to the scalping screen and, hence, through the plant or it may be subjected to an independent screening and sizing operation. Which procedure is used will depend upon whether the crushed material is salable as a separate product or merely represents an economical disposition of oversize. In some cases, the crushed gravel is used

for blending with natural gravel to meet specifications which put a minimum limit on quantity of crushed particles.

Washing

Washing and sizing of sand and gravel are almost always done simultaneously. Water is needed for efficient screening and, of course, is essential to water classification. Accordingly, whether or not washing is needed, water is introduced into the sizing operation if it is available. Dry sizing, involving screening and air separation, has been used in some cases where water is scarce. For sands and gravels containing loosely adhering and readily soluble or suspendable impurities, the minimum amount of water required for sizing is adequate for washing.

However, many sands and gravels contain clay, mud, mudballs, and organic impurities firmly enough attached to require thorough washing and scrubbing. Depending upon circumstances, adequate washing will result from the following, listed (roughly) in order of increasingly difficult conditions: (a) pumping material to the plant; (b) introducing enough water to facilitate screening; (c) providing adequate sprays on vibrating or revolving screens; (d) passing material through special sections of revolving screens in the presence of counter-flowing high-velocity water; (e) same as (d) except that, in effect, the revolving screen is "blinded" (generally called a scrubber) and, perhaps also equipped with abraders in the form of chains or balls; and (f) passing through screws or log washers.

In short, the problem is to grind the sand and gravel together in the presence of adequate water which is replaced frequently enough to carry away the impurities in suspension or in solution. A rough, order-of-magnitude rule is that a minimum of about 10 gal. of water per min is required for each ton of material produced per hr (600 gal. per ton). That is to say, a plant producing 500 tons of sand and gravel per hr (moderately large) would require about 5000 gal. of water per min.

In general, washing presents no insurmountable obstacles, although there are exceptions. Taggart[13] and the *Pit and Quarry Handbook*[12] describe the various processes in considerable detail. Occasionally, organic impurities, as revealed by the colormetric test, are so firmly attached and in such form that they cannot be removed by washing. The damage that they cause, if any, has not been well demonstrated. There is evidence that early strengths may be reduced somewhat but that later strengths are relatively unaffected.

Screening gravel

The efficient use of screens is restricted to the coarser sizes, generally about ¼-in. openings or larger although, infrequently, sizes as small as No. 10- or No. 20-mesh are used. The scalping screen, through which the raw material is introduced to the plant, may be a gravity screen (perhaps only a "grizzly" made of steel bars), a revolving screen, or a vibrating screen. It removes oversize material which is discarded or diverted to a crusher. The undersize

from the scalping screen, or from one immediately following it, is then processed through various screens to produce gravel of required sizes and to separate the sand.

Screening of gravel and the crushed oversize presents no special problems. Vibrating screens are used in the vast majority of cases, although revolving screens, either conical or cylindrical, are sometimes indicated because of their aid in cleaning the material. References 12 and 13 (*Pit and Quarry Handbook* and Taggart) previously cited contain excellent information on screens as applied to gravel.

The range in sizes from nominal minima to nominal maxima, together with feasible tolerances for over and undersizes, are well outlined in *Simplified Practice Recommendations for Sizes of Coarse Aggregate* promulgated by the U. S. Department of Commerce.[14] Closer sizes and tolerances are not considered to offer advantages commensurate with the greatly increased cost of processing. The Simplified Practice Recommendations form the basis for grading specifications for coarse aggregate promulgated by the American Society for Testing Materials, the American Assn. of State Highway Officials, many state highway departments, and other authoritative and representative specification-writing bodies.

It should be pointed out that processing screens and testing sieves do not have the same clear openings for a given nominal size. Testing sieves have square openings with wire diameter and clear openings conforming to universally accepted specifications.[15] Processing screens may be square, rectangular, round, or of other shape. They may be made of woven wire or perforated plate. The only limitation on size or shape of opening of any processing screen should be that, for the nature of feed, conditions of loading, slope, and capacity, it produces the size required. For example, a 1-in. round opening screen in the plant may produce material which, substantially, has a maximum size of ¾ in. as measured by standard testing sieves where sieving is to "refusal." Similarly, a plant screen with square opening of about ¼ in., or larger, may be used to insure a nominal lower size of about No. 4 as measured by testing sieves.

Classifying sand

Classification of sand as to size presents a major problem in the production of sand and gravel. Sand grading, as it affects concrete quality, is more critical than gravel grading and this is particularly true for the leaner mixes. The separation of sand into different sizes does not represent a straightforward screening operation as in the case of gravel. In general, screens are useful only to separate the sand from the gravel and to make required separations coarser than about a 20-mesh sieve. From that point on, sizing is generally by water classification[16]—although, as pointed out earlier, screens, both wet and dry,[17] and air separators are sometimes used.

In the vast majority of cases the sizing of sand consists only of separating it from the gravel and dewatering it in such a manner as to retain, so far

as is feasible, an adequate amount of the finer sizes, generally those finer than a No. 50 sieve. For that purpose there is a wide variety of settling tanks and devices for recovering the sand and leaving the water behind. For example, an inclined screw conveyor in a suitable trough, with an apron overflow at the lower end, is frequently referred to as a screw classifier, a screw washer, or a screw dewaterer. The sand drag, rake classifiers, bowl classifiers, dewatering wheels, and other devices for removing sand from water, with a minimum of turbulence to retain the fines, all accomplish about the same purpose.

Generally, the natural grading of the sand is such that entirely satisfactory results will come from the relatively simple classifying methods referred to. Sometimes, however, for best results more elaborate methods are required. The most authoritative treatments of the general problem of sand classification are by Shaw[16] in his book on sand settling and devices for settling and classifying sand, published by *Rock Products*, and Rockwood[17] in his booklet on screening fine materials, also published by *Rock Products*. Taggart[13] treats hydraulic classification as applied to the mining field in considerable detail. The *Pit and Quarry Handbook*[12] includes a well rounded chapter on "Screens, Classifying and Washing."

The most common difficulty, particularly in water-transported materials, is the absence of fines. In many cases that deficiency has been apparent only and has resulted from inefficient dewatering methods or for other reasons, including loss of fine material during dredging operations. Obviously, if the fines are in the deposit, the solution to their absence in the finished product lies in revising processing methods.

When the fines are not available, the solution is to blend fine materials from other sources (frequently this can best be done at the batching plant) or from crushing larger sizes. Not enough experience has been had with crushing larger sizes for supplying fines to natural sand to permit discussing the problem in specific terms. The tailings from crushers in general, and from impact crushers in particular, have proven helpful. Roll crushers have been used to reduce the smaller sizes of gravel, but not with widespread success. Considerable success has been had with ball mills in grinding sand to finer sizes. Rod mills have been used successfully and economically on some large public works projects where the sand has been derived entirely or principally from the crushing operation.[18]

A common deficiency in the grading of natural sands, particularly those which have been water sorted in nature, is the concentration of an excessive proportion of the grains in the intermediate sizes. It is probably this circumstance which has led to the requirement for excessive fines in concrete sand—in an attempt to overcome, by additional fines, the harshness and tendency to bleed resulting from the high concentrations in the intermediate sizes.

Reducing the proportion of intermediate sizes presents a difficult classifying problem. However, it may be accomplished with varying degrees of

success by the addition of fines as already discussed or by the actual elimination of a portion of the excess sizes. The reduction in quantity of intermediate sizes (generally costly in wasted material) can be accomplished by a variety of classification methods which will not be discussed in detail here. Rising current (of water) classification into different sizes represents the most straightforward approach. The collection and recombination of sizes settled at different points in various systems represents another. A classic treatment of the whole problem of sand classification is contained in the book by Edmund Shaw.[16]

Crushing

Gravel is crushed to reduce oversize, sometimes to produce angular particles and, sometimes, as discussed above, to produce the finer sizes of sand. Another reason, discussed subsequently, is to reduce the quantities of soft and friable particles.

Crushers are divided into five general classes: (1) jaw; (2) gyratory, including cone; (3) roll; (4) impact, including hammermills; and (5) various grinding machines, including rod mills. All of them find use in producing sand and gravel.

Jaw and gyratory crushers are most commonly used as primary crushers for sand and gravel. Secondary and later stage crushers are generally of the gyratory or cone type and sometimes are rolls and hammermills. It is difficult to generalize since the problem of crushing depends so much upon the size of product and purpose of crushing.

Where simple size reduction is the objective, first and second stage and sometimes third stage crushers are used. However, frequently, crushing operations are performed for the principal purpose of eliminating soft particles or providing crushed faces. In such cases the objective is to do a maximum of work with a minimum of size reduction. While any crusher lends itself to this type of operation, with suitable regulation of size and amount of feed in relation to crusher opening, the impact type appears to be best fitted for it.

Crushing gravel involves substantially the same problems as for any material in the mining field, and the literature is voluminous. Fred C. Bond and Frank E. Briber[19] presented an informative paper on "Principles of Crushing" at the 1951 annual meeting of the National Sand and Gravel Assn. Taggart[13] gives much information on the several types of crushers. The *Pit and Quarry Handbook*[12] includes a chapter on "Crushing, Grinding and Separating" in which the application of different types of crushers is discussed. There is further voluminous literature available, but that listed should furnish the necessary fundamental information.

Deleterious particles

Among the more vexing problems of sand and gravel production is that presented by deleterious particles occurring as impurities in deposits. It is

more pressing for gravel than for sand. In general, if a source does not contain basically suitable material, there is little that can be done about it. While there are methods which can be applied to reduce the quantity of many undesirable particles, there is a quickly reached limit beyond which such processing is neither economical nor practicable.

Particles considered deleterious are generally classified as: soft fragments; thin and friable particles; shale; argillaceous sandstones and limestones; porous and unsound cherts; coated particles; thin and elongated particles; laminated particles; coal; lignite; and other materials depending upon impurities prevalent in the locality. These categories may be further condensed into two general classifications so far as use in concrete is concerned—those particles which break down with little volume change and those which expand considerably and exert a disruptive force.

Various attempts have been made to develop processing methods for reducing the quantity of these deleterious particles with only moderately encouraging results. It is self-evident that processing to separate selected types of particles from the parent gravel must depend on differences in strength and hardness, or in weight, or, to a limited extent, in size and appearance, or a combination of these.

Very soft particles, such as mud balls and some loosely consolidated conglomerates, may be removed, or reduced in quantity satisfactorily by processing through log washers or scrubbers. At least one example can be cited of a deposit with the sand and gravel completely embedded in clay, where the high content of sticky clay is removed satisfactorily by log washers. Numerous examples of fairly satisfactory removal of mud balls and the like by scrubbers of various types can be cited. In general, only the simplest of scrubbers are used in the industry.

Particles such as those just discussed, while soft, do not fall in the category of those commonly identified as soft. The common category suggests separation by differences in hardness, which has led to the use of various types of impact crushers and "soft stone eliminators." These have been used with varying success depending upon the material being processed. Their use introduces additional problems: (a) Inevitably, a bottle neck in the flow sheet is produced; (b) What shall be done with the crushed soft materials? Shall they be permitted to go into the sand? In many cases that probably would not be harmful but in others it might; (c) The quantity of soft materials cannot be reduced satisfactorily without, at the same time, breaking up considerable quantities of good materials; (d) Equipment maintenance costs are generally high.

Separation on the basis of weight found little application in the gravel industry until recently. Some simple jigs have been used to separate coal, lignite, and sticks. References to the use of jigs for removal of shale and soft stone are in the literature. Taggart[13] describes various types of jigs in some detail. Jigs require a greater difference in weight than generally exists

between the parent gravel and the material which it is wished to remove, except for such materials as coal and sticks where they may be used quite effectively.

Heavy-media separation (*i.e.*, sink-float), developed in the mining industry within the past 15 years, has proven successful for making separation of materials differing in specific gravity. It is claimed that a range in specific gravity of as little as 0.02 makes separation feasible and that for a range of 0.05, separation can be made quite readily.

In principle the process consists of floating out the lightweight material on a heavy "liquid" which is formed by suspension of heavy materials such as magnetite, galena, or ferrosilicon in water. Quartz sand suspensions have been used for cleaning coal for many years—the Chance process. The maximum specific gravity obtainable with sand, according to Taggart,[13] is only about 1.8, which, of course, would not be applicable to aggregate processing. Again according to Taggart,[13] magnetite will provide suspensions of specific gravities up to 2.55 and galena and ferrosilicon up to about 3.3 for normal working conditions.

Modern heavy-media processes employing ferrosilicon, galena, or magnetite have been used quite widely in processing low-grade iron ore, cleaning coal, and concentrating zinc, feldspar, magnetite, manganese, and other ores.[20,21,22,23] Cases of its use in processing gravel are, as yet, relatively few. The earliest was described in the April, 1949, issue of *The Engineering Journal* (the journal of the Engineering Institute of Canada),[24] the December, 1949, issue of *Pit and Quarry*,[25] the February, 1950, issue of *Rock Products*,[26] and the December, 1951, issue of *Civil Engineering*.[27] Those articles describe separation of gravel for use in the construction of airport runways at Rivers, Manitoba, Canada. The gravel contained considerable quantities of shale and rotten granite and it is said that a satisfactory separation was made at an estimated operating cost of about 10 cents per cu yd. It appears that this figure does not include plant "write-off" or any royalty payments.

Heavy-media installations have been made in at least three commercial gravel operations and these have apparently accomplished their intended purpose of removing objectionable lightweight constituents. Obviously, economy and the nature and amount of deleterious substances will limit to relatively few the plants which can advantageously employ heavy-media processing. Discussions of the application of that process will be found in references 28, 29, and 30.

The efficacy of hand-picked methods should not be overlooked. "Belt-pickers" are familiar in the industry. Commonly, no more special provision is made than to provide a place for a man to stand or sit where he can reach the moving belt. Sometimes a picking table, in the form of a widened and slowed-down section of belt, is provided. More elaborate devices are available, but not commonly used in sand and gravel processing. A good job can be done in the removal of certain impurities, (clay, chert, and large pieces of rotten stone) by alert workmen.

MISCELLANEOUS

It should be clear from the preceding that, although only relatively simple processes are involved, the production of sand and gravel involves possibilities for so many approaches and combinations of approaches as to become a complex subject. In anything less than a comprehensive and voluminous treatise, only the most general aspects of the problems can be touched upon. There are a number of subjects deserving discussion which are not readily classified and which will be discussed briefly in this "catch-all" miscellaneous section.

Breakage in handling

Handling and stockpiling can result in a great deal of undoing of the plant processing. Breakage and segregation may result in significant changes in grading.

Gravels are heavy materials and any handling results in breakage. Accordingly, after screening, care should be taken to minimize the impact due to falling to the extent practical. Excessively high drops should be avoided. Where it is unavoidable that the conveyor discharge is at considerable height above the point of deposit, breakage can be minimized by the use of rock ladders[31] or other means of interrupting the free drop. Sometimes breakage due to handling is enough that rescreening and rinsing are required. It is not uncommon to rescreen and rinse gravel as it is loaded into cars and other conveyances. The best practice, however, is to handle the material so that this additional processing will not be required.

Stockpiling

Segregation is a problem of particular importance with gravel. Sand is not free from it but, since it is generally being handled moist, little segregation is likely to take place. The importance of segregation of gravel is diminished as the sizes are more closely graded. It is very difficult, almost impossible, to handle "long" gradings (No. 4 to 2 in. for example) without objectionable segregation while with short gradings ($\frac{1}{2}$ to 1 in. for example) segregation which takes place may be relatively unimportant.

Aggregates should be handled in stockpiles and bins in a manner to minimize segregation of sizes. "Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)"[32] contains some valuable suggestions for handling materials. Reference should also be made to the Bureau of Reclamation's *Concrete Manual* and to the *Pit and Quarry Handbook*[12] chapter on "Open Storage and Reclamation."

Reclaiming

Material in stockpiles must be reclaimed for transportation to point of use. There are numerous types of portable loaders for this purpose[12] and conventional cranes are adaptable. Stockpiling over a tunnel with reclaiming by belt offers many advantages including that of the ready blending of different sizes of materials.

Disposal of slimes

Spent wash water generally contains considerable quantities of clay and other fine material in suspension. Frequently circumstances, including stream pollution regulations, make the disposal of this material a problem—that of settling out the solids before disposing of the water or disposing of the suspension in such a manner as to be unobjectionable.

This problem only recently has become a prominent one in the sand and gravel industry, brought about by increasing attention to stream pollution. The best procedures for handling it have not as yet been worked out. It is mentioned here as something which the designer of plant and appurtenances should have in mind. Settling ponds and filters appear to represent the most direct approach. Re-using the clarified wash water frequently represents an economy.

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Ultimate Strength in Shear of Simply-Supported Prestressed Concrete Beams Without Web Reinforcement*

By E. M. ZWOYER† and C. P. SIESS‡

SYNOPSIS

Tests were made on 34 simply-supported prestressed concrete beams without web reinforcement. All beams were rectangular in cross section and were reinforced with straight wires bonded throughout their length. Variables included the prestress, the percentage and type of steel, the concrete strength, and the ratio of shear-span to depth of beam. On the basis of the results obtained, an hypothesis of failure was developed which is consistent with the phenomena observed in the tests and which provides the basis for a method of analysis capable of predicting the strength in shear of beams similar to those tested. This method of computing shear strength can be applied also to beams of ordinary reinforced concrete, and comparisons with the results of tests show reasonably good agreement.

INTRODUCTION

Because of the sudden and relatively brittle nature of shear failures, most engineers and writers of specifications are agreed that this type of failure should be avoided. Before this can be done, however, we must have at our disposal a method of computing the shear strength of a beam in terms of the many variables which may affect its behavior. This is a complex problem and one involving a great many variables, even for ordinary reinforced concrete beams, while the addition of prestress introduces further complications because of its effect on the formation of cracks due to diagonal tension stresses.

The tests and analyses reported in this paper represent the initial stages of a more extensive investigation aimed at the over-all problem of shear strength of prestressed concrete beams. This first step was intended primarily to increase our understanding of the phenomena and mechanisms of shear failure, and to provide basic data which could be used in planning additional tests involving other variables.

TESTS

Description of beams

All of the beams tested were 6 in. wide, 12 in. over-all depth, and 10 ft long. Beams S-21 through S-27 were tested on a 9-ft span with a single load at midspan; beams S-29 through

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S-32 were tested on a 7-ft span with two loads, 15 in. each side of midspan; and all other beams were tested on a 9-ft span with two loads 15 in. each side of midspan. Dimensions of the individual beams are given in Table 1.

Reinforcement consisted of cold-drawn wires having a measured diameter of 0.193 or 0.199 in. Typical stress-strain curves for the two types of wire used are shown in Fig. 1. Thirty-two of the beams (S-1 through S-32) were post-tensioned. For these beams, a 3 x 2-in. hole was formed with its center about 8 in. below the top of the beam. After the reinforcement was placed and tensioned, this hole was completely filled with a sand-cement grout to provide bond. A small amount of aluminum powder was added to the grout to minimize shrinkage and prevent the loss of bond. The post-tensioned beams were tested only two to three days after tensioning and grouting to minimize losses and thus provide better control of the prestress force. Compressive strength of the grout at the time of testing ranged from 2000 to 5000 psi as determined from cylinder tests.

Two pre-tensioned beams (S-33 and S-34) were tested to determine if the rather large area of the grout core had any effect on the results obtained. No such effect was noted.

Properties of the individual beams are listed in Table 1. Concrete strengths varied from 2500 to 8000 psi, with the majority falling between 3000 and 6000 psi. The amount of tension reinforcement was varied from 0.27 to 0.93 percent, including both under-reinforced and over-reinforced beams. Initial prestress in the steel was varied from 5000 to 150,000 psi, with most beams having prestresses of either 60,000 or 120,000 psi. The effective prestress f_{se} at the time of test was somewhat lower than the initial prestress and the value for each beam is given in Table 1. The ratio of shear span a to effective depth d was varied from 2.8 to 6.7.

Measurements

Strains were measured on the steel reinforcement with Types A7 and A12 SR-4 electric strain gages; on the top surface of the beam with Types A9 and A11 SR-4 gages; and on the

TABLE 1—PROPERTIES OF PRESTRESSED CONCRETE BEAMS

Beam No.	Concrete strength f_c', psi	Beam width b, in.	Eff. depth d, in.	Reinforcement			$Q' = \frac{E_s p}{k_1 k_3 f_c'}$	Effective prestress f_{se}, psi	Beam span L, in.	Shear span a, in.	a/d
				Type	Area A_s, sq. in.	p, percent					
S-1	3620	6.00	9.16	I	0.176	0.320	21.4	10000	108	36	3.9
S-2	3660	6.00	9.28	I	0.176	0.316	21.4	50000	108	36	3.9
S-3	4290	6.00	9.38	I	0.176	0.312	18.3	24000	108	36	3.8
S-4	3085	6.00	8.00	I	0.440	0.916	69.6	116000	108	36	4.5
S-5	6260	6.00	8.30	I	0.440	0.883	40.3	103400	108	36	4.3
S-6	7990	6.00	8.20	I	0.440	0.893	36.1	110000	108	36	4.4
S-7	3550	6.00	8.44	I	0.440	0.868	58.7	104300	108	36	4.3
S-8	6120	6.13	8.20	I	0.381	0.758	35.0	5000	108	36	4.4
S-9	4760	6.00	8.20	I	0.381	0.774	42.1	56800	108	36	4.4
S-10	5790	6.00	8.20	I	0.381	0.774	36.8	72000	108	36	4.4
S-11	2580	6.00	8.80	I	0.176	0.333	29.3	36100	108	36	4.1
S-12	4760	6.00	8.20	I	0.381	0.774	42.0	30400	108	36	4.4
S-13	4840	6.00	8.20	I	0.381	0.774	41.3	140000	108	36	4.4
S-14	4660	6.00	8.20	I	0.352	0.715	39.4	131400	108	36	4.4
S-15	2880	6.00	8.35	I	0.176	0.351	28.2	88000	108	36	4.3
S-16	3530	6.00	8.06	I	0.176	0.364	24.8	89400	108	36	4.5
S-17	3350	6.00	8.81	I	0.352	0.665	48.4	136000	108	36	4.1
S-18	4150	6.00	8.31	I	0.234	0.470	28.3	59000	108	36	4.3
S-19	3850	6.00	8.38	I	0.176	0.350	22.2	60000	108	36	4.3
S-20	5350	6.00	8.45	I	0.176	0.347	17.4	61200	108	36	4.3
S-21	6220	6.00	8.24	I	0.440	0.889	39.3	116000	108	54	6.6
S-22	5630	6.00	8.12	II	0.467	0.958	46.4	59100	108	54	6.7
S-23	4360	6.00	8.02	II	0.373	0.776	44.9	124500	108	54	6.7
S-24	2900	6.00	8.41	II	0.467	0.924	74.1	116000	108	54	6.4
S-25	2900	6.00	8.44	II	0.249	0.491	39.4	114000	108	54	6.4
S-26	3130	6.00	8.95	II	0.218	0.405	30.4	58900	108	54	6.0
S-27	3350	6.00	8.45	II	0.156	0.307	21.8	61100	108	54	6.4
S-28	3470	6.00	8.80	I	0.147	0.277	19.1	58000	84	36	4.1
S-29	3320	6.00	8.53	II	0.311	0.608	43.5	117000	84	24	2.8
S-30	3350	6.00	8.50	II	0.249	0.488	34.8	118000	84	24	2.8
S-31	2440	6.00	8.42	II	0.280	0.554	51.6	117900	84	24	2.8
S-32	3350	6.00	8.35	II	0.218	0.435	30.9	117000	84	24	2.9
S-33	3400	6.00	8.60	II	0.311	0.603	42.3	108310	108	36	4.2
S-34	5800	6.00	8.64	II	0.311	0.600	28.6	114000	108	36	4.2

sides of the beams at the level of the steel with a 36-in. gage-length direct reading gage employing a 0.001-in. dial indicator.

Deflections were measured with 0.001-in. dial indicators at midspan and under the load points on all beams and also at the one-third points of the span on the beams loaded at midspan.

Dynamometers consisting of aluminum cylinders with electric strain gages attached were placed at one end of each reinforcing wire and were used to measure the initial prestressing force and that portion of the loss in prestress occurring before the grout had set fully.

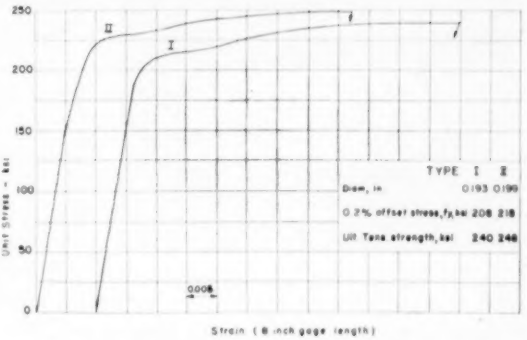


Fig. 1—Stress-strain curves for wire reinforcement

MANNER OF FAILURE OF TEST BEAMS

Before cracking of the concrete, all of the beams showed essentially "elastic" behavior. Deformations were linearly proportional to load, and the load could be released and reapplied without changing the behavior of the beams. The presence of prestress increased the cracking load and thus extended the range of elastic action.

As the load was increased, cracks formed and the deformations of the beam began to increase more rapidly than the load. These cracks were vertical in the region of pure flexure between the loads but were inclined in the typical manner in the regions subjected to shear as well as flexure (Fig. 2).

Final failure occurred in all cases by crushing of the concrete in the compression zone of the beam. Four of the beams failed initially in flexure. In these beams, crushing occurred in the central portion of the beam over fully-developed flexure cracks and was preceded by "yielding" of the reinforcement. A fifth beam failed in flexure by crushing before the steel had yielded, but this was the result of the reinforcement being unbonded over the length of beam between the loads.

The remaining beams failed in shear. In these beams, the inclined cracks due to diagonal tension developed much more rapidly than the vertical

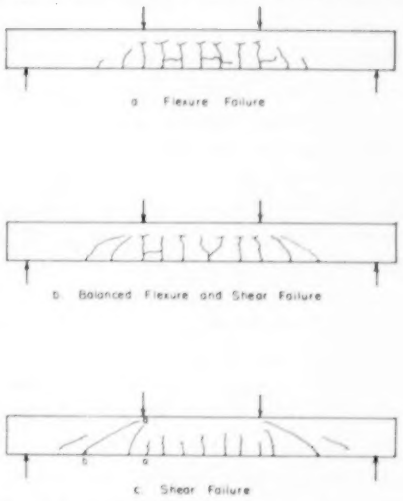


Fig. 2—Typical patterns of cracking at ultimate loads

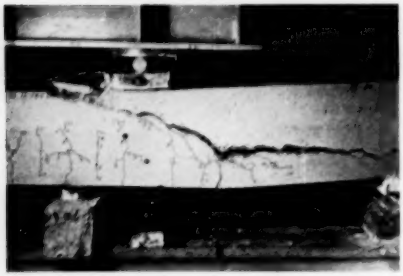


Fig. 3—Typical beam after failure in shear

cracks in the region of pure flexure. Final crushing occurred, therefore, over one of the inclined cracks and adjacent to a load point. As shown in Fig. 2(c), these inclined cracks extended much higher than the vertical cracks and thereby reduced the depth of the compression zone at the location where crushing occurred. Near failure, the principal inclined cracks on each side of the beam extended almost horizontally under the load point and invaded the "pure moment" region of the beam. In most cases, the crushing occurred on the midspan side of the load point, as shown in Fig. 3. Splitting of the beam along a horizontal plane at the level of the reinforcement also usually occurred at failure (Fig. 3). However, on the basis of observations made during the tests, it is believed that this splitting was a secondary effect, occurring after the destruction of the compression zone of the beam and having no significant effect on the ultimate strength in shear.

Although both types of beams failed finally by crushing of the concrete in the compression zone, there were certain differences between the beams failing in flexure and in shear. For flexure failures, crushing occurred over a vertical crack after the reinforcement had yielded; in other words, these were typical tension failures. For shear failures, however, crushing occurred over an inclined crack and the reinforcement was well below yielding except for cases of nearly "balanced" failure.

On the basis of these observations, the concept was developed that shear failures could be considered as a type of premature compression failure resulting from the greater height reached by the inclined cracks as compared to the vertical cracks caused by purely flexural stresses. Measurements of strains in the concrete on the top surface of the beam showed that these strains were usually quite small except in the region where final crushing occurred. At this location, the concrete strains were in good agreement with those usually measured at crushing of beams failing in flexure. It can be seen from Fig. 2 that the inclined cracks are directed toward the load point, and the strains in the concrete tend to be "concentrated" in the region near the upper ends of the inclined cracks, that is, near the load point. This concentration of strain, together with the greater height of the inclined cracks as compared to normal flexure cracks, effectively weakens the compression zone of the beam and leads to a premature failure in compression.

A significant corollary to the concept of shear failure as a premature or modified compression failure is that failure occurs at some limiting moment rather than at some limiting value of the shear itself. The effect of shear in the beam is to cause the formation and growth of inclined cracks which

reduce the depth to the neutral axis and thereby reduce the capacity of the compression zone to a value below that for a similar beam failing in a region of pure flexure.

For the beams tested in this investigation, it has been possible to derive expressions for the limiting moment corresponding to the ultimate strength of a beam failing in shear. Since these expressions involve empirical coefficients derived from a limited number of tests, they must be considered applicable only to prestressed concrete beams similar to those tested. Nevertheless, the successful use of the derived expressions to predict the strength of beams failing in shear is believed to substantiate the concept of compression failure at a limiting moment. It is important to note, however, that an expression for the limiting shear-moment is not in itself sufficient to determine whether a given beam will fail in flexure or in shear. The shear-moment expression applies only to beams in which inclined cracks have developed to such an extent that the capacity of the compression zone of the beam is impaired. Consequently, if the ratio of moment to shear is high, as in the case of long spans, a flexural failure may occur before inclined cracks have had a chance to develop. In such a case, the capacity of the beam would be determined by the ultimate flexural moment, which would be greater than the limiting shear-moment. The tests described in this paper provide no data regarding the effect of moment-shear ratio on the development of inclined cracks and the final mode of failure of a prestressed concrete beam.

DERIVATION OF EXPRESSIONS FOR ULTIMATE MOMENT CAPACITY OF BEAMS FAILING IN SHEAR

The analysis of beams failing in shear is based on the following assumptions which are essentially the same as those made in the derivation of expressions for beams failing in flexure:

(a) It is assumed that the maximum capacity of the beam is reached when the concrete crushes at some limiting value of strain, determined from these and other tests to be about 0.004.

(b) Tension stresses in the concrete below the neutral axis are neglected.

(c) The stress-strain relationship for the steel reinforcement is assumed to be known.

(d) The distribution of compressive stress in the concrete at ultimate load is defined by three parameters, k_1, k_2, k_3, as shown in Fig. 4. The values to be assigned to the parameters have been determined empirically from the tests, or have been estimated as described subsequently.

(e) It is assumed that the reinforcement is bonded to the concrete throughout its length.

The notations used throughout the following derivations are defined at the end of this paper.

From Fig. 4, the resisting moment of the beam at ultimate load is obtained from a condition of equilibrium as

$$M_{ult} = Td\,(1 - k_2k_u) = f_{su}\,pbd^2\,(1 - k_2k_u) \quad \ldots\ldots (1)$$

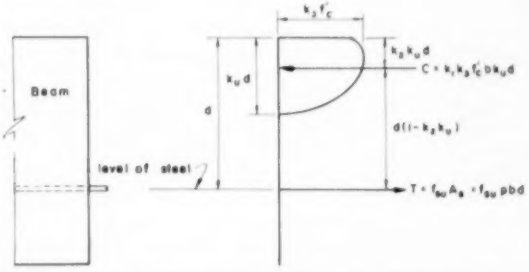


Fig. 4—Internal forces at section of failure in shear

A second equation of equilibrium requires that the total tension equal the total compression:

$$T = C \quad (2)$$

or

$$f_{su}\, pbd = k_1 k_3 f_c' bk_2 d \quad (2a)$$

from which

$$k_u = \frac{f_{su} p}{k_1 k_3 f_c'} \quad (3)$$

The following expression for ultimate moment is obtained by substituting Eq. (3) into Eq. (1):

$$M_{ult} = f_{su}\, pbd^2 \left(1 - \frac{k_2 f_{su} p}{k_1 k_3 f_c'}\right) \quad (4)$$

The quantities b, d, and f_c' in Eq. (4) are physical properties of the beam and are assumed to be known. The quantities k_1, k_2, k_3 are properties of the concrete stress block (Fig. 4) and for the time being are unknowns. Subsequently, however, the term k_2 is assigned an arbitrary value, and the product $k_1 k_3$ is evaluated empirically from the results of the tests. The other term appearing in Eq. (4) is f_{su}, the ultimate steel stress, which is unknown and which is, of course, the principal variable affecting the ultimate moment.

Eq. (4) is based only on the equations of statical equilibrium and is valid for any rectangular beam irrespective of the mode of failure. Its use for flexural and shear failures differs only in the method used to determine the value of the steel stress f_{su} at the moment corresponding to crushing of the concrete.

To derive an expression for f_{su}, recourse must be had to the deformations of the beam; that is, to the strains in the concrete and the steel. Total steel strain at ultimate load at the section of failure is most conveniently expressed as the sum of three parts: (1) The steel strain due to prestress alone is designated as ϵ_{se}, and the compressive strain in the concrete at the level of the steel, at the same stage, is ϵ_{ce}. (2) As load is applied to the beam, the compressive strain in the concrete at the level of the steel decreases, and at some stage of loading it reaches zero. At this stage, the steel strain has increased to $\epsilon_{se} + \epsilon_{ce}$. (3) With further increase in load, the compressive strain in the concrete at the top of the beam increases until failure occurs by crushing at a concrete strain ϵ_u. During this stage, the concrete at the level of the steel is strained in tension by an amount designated as ϵ_{cu}, and the steel is assumed to undergo an equal strain. Thus, the total steel strain at ultimate load may be expressed as

$$\epsilon_{su} = \epsilon_{se} + \epsilon_{ce} + \epsilon_{cu} \quad (5)$$

The terms ϵ_{se} and ϵ_{ce} in Eq. (5) are known or can be computed easily, as described later. However, before this equation can be solved for the ultimate steel strain ϵ_{su}, the concrete strain ϵ_{cu} at the level of the reinforcement must be determined. In the analysis of beams failing in pure flexure, it is usually possible to relate the concrete strain at the level of the steel to the known ultimate concrete strain ϵ_u at the top of the beam by assuming a linear distribution of strain in the concrete throughout the depth of the beam. However, the presence of inclined cracks as shown in Fig. 2(c) makes it unreasonable to assume a linear distribution of strain at the section of failure in beams failing in shear. It becomes necessary, therefore, to resort to empirical, and approximate, methods to obtain expressions for the quantity ϵ_{cu} which is needed to determine ϵ_{su} and thus f_{su}.

Although strains at the section of failure cannot be considered to vary linearly with depth, it has been assumed that such linear variation will apply to the average strains in the concrete over some length of the beam extending from the section of failure near the load point outward to a point beyond the region in which the inclined cracks are found. Neither the exact length of this portion of the beam nor the distribution of strain over this length can be determined from available data, and herein lies the approximate and empirical nature of the expressions which have been derived.

For the conditions at ultimate load, the average concrete strain on the top surface of the beam over some length is defined in terms of the ultimate strain at the section of failure as $K_1 \epsilon_u$. Similarly, the average strain at the level of the steel over the same length of beam is

defined in terms of the strain at the section of failure as $K_2\epsilon_{cu}$. If these average strains are assumed to vary linearly with depth, then

$$\frac{K_1\epsilon_u}{k_u d} = \frac{K_2\,\epsilon_{cu}}{d - k_u d} \qquad (6)$$

and

$$\epsilon_{cu} = \frac{K\,\epsilon_u\,(1 - k_u)}{k_u} \qquad (6a)$$

where

$$K = \frac{K_1}{K_2}$$

The quantity K is an empirical parameter involving the unknown distribution of concrete strains along the beam at both levels as well as the unknown or undetermined length of beam over which the strains have been averaged. This term has been evaluated from the test results as described subsequently.

Substituting k_u from Eq. (3) in Eq. (6a), one obtains

$$\epsilon_{cu} = K\,\epsilon_u\left(\frac{k_1 k_2 f_c'}{f_{su}p} - 1\right) \qquad (6b)$$

By substituting Eq. (6b) into Eq. (5), the following expression can be found for the ultimate steel stress:

$$f_{su} = \frac{K\epsilon_u\,k_1k_2f_c'/p}{\epsilon_{su} - \epsilon_{se} - \epsilon_{cc} + K\epsilon_u} \qquad (7)$$

The absolute value of the strain should be used in Eq. (7) since the signs have been taken into account in the derivation.

The quantities p and f_c' in Eq. (7) are known as properties of the beam. The strain ϵ_{se} in the steel can be determined from the stress-strain diagram for the steel for any given effective prestress f_{se}. The strain ϵ_{cc} in the concrete at the level of the steel, due to f_{se}, can be computed by an elastic analysis for the uncracked section. If the term $K\epsilon_u$ and the product k_1k_2 are obtained from the empirical expressions derived subsequently, the only unknown term remaining on the right hand side of Eq. (7) is the steel strain at ultimate ϵ_{su}. Eq. (7) may then be considered to define a relationship between steel stress f_{su} and steel strain ϵ_{su} at ultimate load. Another such relationship between steel stress and strain is represented by the stress-strain diagram for the steel, and only one combination of stress and strain will satisfy both of these conditions simultaneously. In these studies, the value of f_{su} at ultimate load was determined by a trial-and-error process involving Eq. (7) and the actual stress-strain diagram for the reinforcement used in the test beams.

Calculation of the ultimate moment is seen to involve two steps: (1) determination of f_{su} from Eq. (7) and the stress-strain diagram for the reinforcement, and (2) substitution of this value of f_{su} in Eq. (4). Before this can be done, however, in any given case, it is necessary to know the values of the terms k_1, k_2, k_3, and $K\epsilon_u$ which appear in Eq. (4) and (7). Evaluation of these quantities is described in the following sections.

EVALUATION OF PARAMETERS

Evaluation of k_2

The quantity k_2 defines the location of the resultant compressive force in the concrete, as shown in Fig. 4. If the steel stress f_{su}, the depth to the neutral axis $k_u d$, and the ultimate moment M_{ult}, are all known at failure, it is theoretically possible to determine k_2 from the following expression, derived by statics from Fig. 4:

$$k_2 = (f_{su}\,pbd^2 - M_{ult})/(k_u\,f_{su}\,pbd^2)$$

In practice this equation cannot be used successfully since it involves a small difference of two relatively large quantities in the numerator. Small errors in either f_{su} or M_{ult} consequently lead to large errors in k_2. This situation, however, suggests that small variations in k_2 will

have little effect on the ultimate moment calculated from Eq. (4) and this is of course true. Since a reasonable value for k_2 must lie between 0.33 for a triangular stress block and 0.50 for a rectangular stress block, and since its effect on M_{ult} is small, it was considered sufficiently accurate to assign an arbitrary average value of 0.42 to k_2 for use in all subsequent calculations.

Evaluation of $k_1 k_3$

The terms k_1 and k_3 always appear as the product $k_1 k_3$ and have therefore been evaluated in that form as follows. Taking moments about the level of the steel in Fig. 4

$$M_{ult} = k_1 k_3 f_c' b k_u d (1 - k_2 k_u) d$$

from which

$$k_1 k_3 = \frac{M_{ult}}{(1 - k_2 k_u) f_c' b k_u d^2} \quad (8)$$

For each beam tested, the value of $k_1 k_3$ was obtained by means of Eq. (8) using $k_2 = 0.42$, the observed moment at failure M_{ult}, and a value for k_u based on the depth to the neutral axis as measured in the test. Values of $k_1 k_3$ obtained from this calculation are listed in Table 2 and plotted in Fig. 5 as a function of the concrete cylinder strength f_c'. The following linear equation was fitted to the points in Fig. 5:

$$k_1 k_3 = 1.500 - 0.715 \times 10^{-4} f_c' \text{ (psi)} \quad (9)$$

This equation is probably not valid for concrete strengths higher than those obtained in the tests since it is believed that $k_1 k_3$ should approach some lower limiting value as the concrete strength increases. Similarly, there is no basis in these tests for using Eq. (9) for concrete strengths below those shown in Fig. 5. In any case, it was felt that the scatter in the plotted points did not justify the use of a more refined empirical equation.

Evaluation of $K\epsilon_u$

The parameter K introduced in Eq. (6a) always appears as a part of the product $K\epsilon_u$, and it is this product which has been evaluated empirically. From Eq. (7), one obtains

$$K\epsilon_u = \frac{\epsilon_{su} - \epsilon_{se} - \epsilon_{ce}}{\left(\frac{k_1 k_3 f_c'}{f_{su} p} - 1\right)} \quad (10)$$

In Eq. (10), the terms p and f_c' are known, ϵ_{se} and ϵ_{ce} can easily be computed for a known effective prestress, and $k_1 k_3$ can be obtained from Eq. (8). The quantity f_{su} was obtained from Eq. (1) using measured values of M_{ult} and $k_u d$ and $k_2 = 0.42$. Once f_{su} was known, the strain ϵ_{su} could be determined from the stress-strain diagram for the reinforcement.

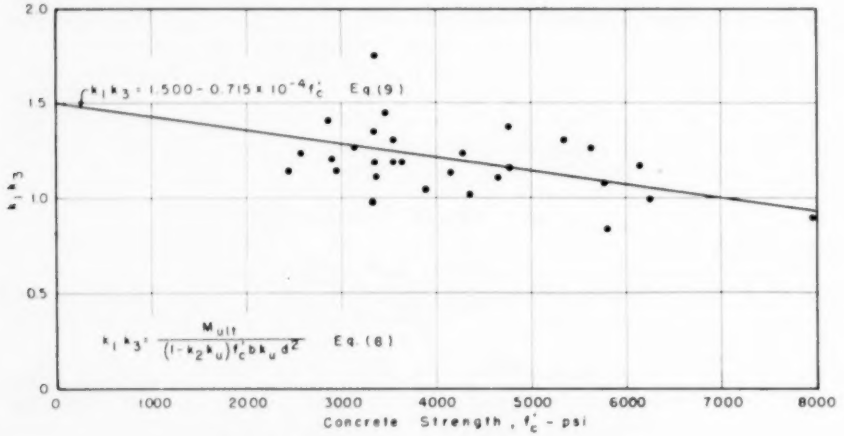


Fig 5—Relation between $k_1 k_3$ and concrete strength

Values of $K\epsilon_u$ computed from Eq. (10) are given in Table 2 and plotted in Fig. 6 as a function of the effective prestress f_{se}. Studies of the data indicated that $K\epsilon_u$ did not vary in any consistent fashion with p, f_c', or the shear-span ratio a/d. The only consistent variation was with the effective prestress f_{se}, and a line having the following equation was fitted through the points in Fig. 6.

$$K\epsilon_u = 0.00040 + 5 \times 10^{-9} f_{se} \text{ (psi)} \quad (11)$$

The rather considerable scatter of the points in Fig. 6 is probably due in part to variations in the ultimate concrete strain ϵ_u, which has usually been observed to vary over a considerable range. Correlation between Eq. (11) and the plotted points in Fig. 6 is admittedly rather poor, and additional test data would probably lead to a different empirical expression. It is also necessary to point out that the relation expressed by Eq. (11) probably applies only to rectangular beams, and certainly only to bonded reinforcement. The use of other shapes of cross section would change the pattern of cracking and the strain distribution in the concrete and would thus affect the factor K.

COMPARISON OF MEASURED AND COMPUTED DEPTH TO THE NEUTRAL AXIS

The method of computing the ultimate moment for beams failing in shear that has been described in the preceding portion of this paper involves two parameters, k_1k_3 and $K\epsilon_u$, which have been evaluated empirically. Since two factors had to be determined empirically, it was necessary to utilize two separate measured quantities from the tests: ultimate moment and depth to the neutral axis. If, for example, only data regarding ultimate moment had been available, it would still have been possible to choose values of k_1k_3 and $K\epsilon_u$ which when used together would have been capable of predicting ultimate moments. However, these values would not necessarily yield correct predictions of the depth to the neutral axis at failure, whereas the expressions derived in this paper should.

For each beam tested, the depth to the neutral axis k_ud was computed with the aid of Eq. (9), (11), (7), and (3). Measured values of k_ud were obtained by measuring with a scale the distance from the top of the beam

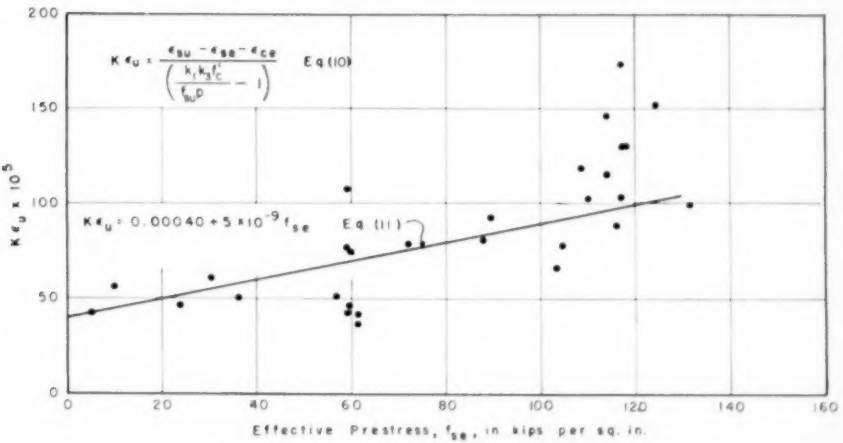


Fig. 6—Relation between $K\epsilon_u$ and effective prestress

TABLE 2—RESULTS OF PRESTRESSED CONCRETE BEAM TESTS

Beam No.	k_1k_3, Eq. (8)	$Ke_u \times 10^3$, Eq. (10)	k_u			Ult. load P_{ult}, lb	$M_{ult}/k_1k_3f_c'bd^2$			Nominal shear $v = \frac{V_{ult}}{bjd}$ psi	$\frac{v}{f_c'}$	Max. principal tens. stress at design load S_t, psi	$\frac{S_t}{f_c'}$
			Meas.	Comp.	Meas./Comp.		Meas.	Comp.	Meas./Comp.				
S-1	1.190	56	0.109	0.096	1.133	12475	0.099	0.092	1.076	122	0.034	—	—
S-2*	0.750	216	0.245	0.126	—	17370	0.123	—	—	167	0.046	—	—
S-3	1.237	47	0.094	0.101	0.933	14040	0.094	0.097	0.969	133	0.031	—	—
S-4†	—	—	—	—	—	21500	0.256	—	—	251	0.081	20	0.007
S-5	0.995	66	0.241	0.252	0.956	31030	0.205	0.225	0.916	344	0.055	20	0.003
S-6	0.899	103	0.244	0.236	1.034	32990	0.198	0.212	0.934	367	0.046	21	0.003
S-7	1.190	79	0.329	0.327	1.006	28040	0.266	0.281	0.947	309	0.087	22	0.006
S-8	1.165	43	0.113	0.117	0.966	17513	0.118	0.111	1.063	191	0.031	—	—
S-9	1.399	51	0.162	0.199	0.814	22930	0.185	0.182	1.017	257	0.054	—	—
S-10	1.072	78	0.204	0.203	1.005	26390	0.187	0.186	1.005	295	0.051	—	—
S-11	1.239	51	0.138	0.141	0.979	10700	0.122	0.131	0.931	112	0.043	—	—
S-12	1.158	71	0.180	0.166	1.084	20000	0.172	0.155	1.110	235	0.049	—	—
S-13*	0.739	391	0.462	0.281	—	29950	0.239	—	—	336	0.069	23	0.005
S-14	1.101	99	0.276	0.287	0.962	27930	0.233	0.252	0.925	312	0.067	19	0.004
S-15	1.407	81	0.169	0.184	0.919	14700	0.169	0.169	1.000	159	0.055	—	—
S-16	1.311	94	0.160	0.166	0.964	14910	0.156	0.155	1.003	166	0.047	—	—
S-17*	0.789	247	0.300	0.309	—	27000	0.246	—	—	282	0.084	26	0.008
S-18	1.136	36	0.132	0.161	0.820	13730	0.120	0.151	0.795	150	0.036	—	—
S-19	1.040	74	0.158	0.140	1.129	13855	0.125	0.132	0.947	149	0.039	—	—
S-20	1.301	40	0.091	0.116	0.784	14470	0.102	0.110	0.927	152	0.028	—	—
S-21*	0.909	206	0.332	0.263	—	24300	0.248	—	—	271	0.044	15	0.002
S-22	1.272	46	0.172	0.215	0.800	17150	0.189	—	—	195	0.035	—	—
S-23	1.016	152	0.342	0.278	1.230	18620	0.251	0.246	1.020	214	0.049	13	0.003
S-24	1.145	89	0.444	0.415	1.070	18790	0.320	0.342	0.936	209	0.072	17	0.005
S-25	1.216	147	0.282	0.244	1.156	13850	0.234	0.218	1.073	150	0.052	9	0.003
S-26	1.260	109	0.189	0.166	1.139	10860	0.153	0.155	0.987	110	0.035	—	—
S-27	1.762	35	0.089	0.135	0.659	8000	0.119	0.127	0.937	85	0.025	—	—
S-28	1.447	79	0.114	0.125	0.912	14040	0.125	0.118	1.059	141	0.041	—	—
S-29	0.982	174	0.359	0.271	1.325	36250	0.238	0.240	0.992	390	0.117	39	0.001
S-30	1.185	131	0.251	0.226	1.111	31980	0.210	0.204	1.029	342	0.102	31	0.009
S-31	1.141	130	0.356	0.307	1.160	29870	0.262	0.267	0.981	325	0.133	34	0.014
S-32	1.354	103	0.199	0.209	0.952	28950	0.197	0.191	1.031	315	0.094	25	0.007
S-33	1.114	120	0.291	0.254	1.146	24160	0.229	0.226	1.013	258	0.076	16	0.005
S-34	0.831	115	0.246	0.199	1.236	26550	0.172	0.182	0.945	278	0.048	17	0.003

*S-2, S-13, S-17, and S-21 failed in flexure.

†Because of mechanical difficulties encountered during grouting procedure, beam S-4 had no grout around the reinforcement in the center 3 ft of the beam; consequently, the beam failed in compression.

to the level of the crack over which failure occurred. As can be seen from the crack patterns in Fig. 2(b) and 2(c), the level of the crack could be determined with a fair degree of accuracy because of the inclination of the cracks. It was assumed in this measurement that the extent of the tension zone above the crack was small as would probably be the case at ultimate load.

Measured and computed values of k_u are given in Table 2 together with their ratio. The average ratio of measured to computed values for beams failing in shear was 1.02 with a standard deviation of 0.15.

COMPARISON OF MEASURED AND COMPUTED ULTIMATE MOMENTS

The ultimate moments measured in the tests are compared with those computed from Eq. (4) in Table 2. The values tabulated are $M_{ult}/k_1k_3f_c'bd^2$, where the denominator is of course identical for the measured and computed values. The average ratio of measured to computed moment was 0.99 with a standard deviation of 0.063.

Measured and computed moments may also be compared graphically, but for this purpose a dimensionless form of Eq. (4) must be used to present the results for all beams on a single figure. If both sides of Eq. (4) are divided by $k_1k_3f_c'bd^2$, and if k_u is substituted for the term $f_{su}p/k_1k_3f_c'$ in accordance with Eq. (3), one obtains the following dimensionless expression:

$$\frac{M_{ult}}{k_1k_3f_c'\,bd^2} = k_u\,(1 - k_2k_u) \qquad (12)$$

In Fig. 7, this equation is shown by the solid line while the points represent measured values of ultimate moment plotted against the corresponding computed value of k_u. Since the term $k_1k_3f_c'$ appears in the deonominator of both variables, the principal effect of variations in $k_1k_3f_c'$ is indicated by the departure of the plotted curve from a straight line.

The agreement between measured and computed moments is sufficiently good to suggest that Eq. (9) and (11) reflect reasonably well the behavior of the beams in spite of the relatively large scatter of plotted points from the empirical equations in Fig. 5 and 6. The ranges of concrete strength, reinforcement percentage, and prestress represented by the beams tested are indicated in Table 1. In addition to these variables, the ratio of shear-span to depth of beam was varied from 2.8 to 6.7, and over this range of a/d the theory based on failure at a limiting moment rather than a limiting shear seems to be substantiated.

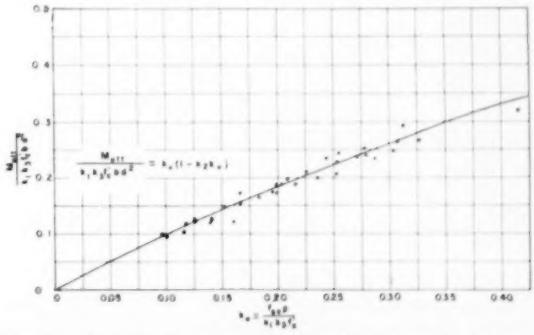


Fig. 7—Comparison of measured and computed ultimate resisting moments using computed f_{su}

EFFECT OF PRESTRESS ON ULTIMATE MOMENT FOR BEAMS FAILING IN SHEAR

It is evident from Eq. (11) as well as from other considerations that the magnitude of the prestress affects the strength of a beam failing in shear. The effect of this variable is easily computed from the expressions that have been derived. For a given cross section and type of reinforcement, and for a particular value of the effective prestress f_{se}, the steel stress at ultimate f_{su} as determined from Eq. (7) can be expressed as a function of the term $p/k_1k_3f_c'$.

Values of ultimate moment for various levels of prestress have been computed for beams similar to those used in these tests and for Type I steel (Fig. 1), and the results plotted in Fig. 8. Use has again been made of a dimensionless plot to reduce the results to a single curve for each value of prestress. The variable $p/k_1k_3f_c'$ has been made dimensionless in this case by introducing, more or less arbitrarily, the modulus of elasticity of the steel E_s to give the parameter Q' in Fig. 8. The curves are cut off at their lower ends at a value of Q' for which the steel stress at failure, found from Eq. (7), was equal to the 0.2 percent offset stress since it appeared that the type of beam tested failed in flexure rather than shear for lower values of Q'.

LIMITS BETWEEN SHEAR AND FLEXURE FAILURES

The dividing line between shear and flexure failures is not always clearly defined by the behavior of the beams under test. However, it is possible to make an arbitrary distinction based on the fact that the ultimate steel stress for beams failing in flexure is always well into the inelastic range whereas the steel stress for beams failing in shear is usually still in the elastic range or at least not much beyond it. Since the type of steel used in these tests did not have a well-defined yield point, the division between shear and flexure failures was arbitrarily placed at the 0.2 percent offset stress for the steel. An attempt to separate shear and flexure failures according to this criterion is illustrated in Fig. 9. Beams falling above the line would have computed steel stresses at failure less than the 0.2 percent offset stress for the particular steel in question and should therefore fail in shear. Conversely, those falling below the line should fail in flexure. The plotted points correspond to the beams included in these tests and in another concurrent investigation.*

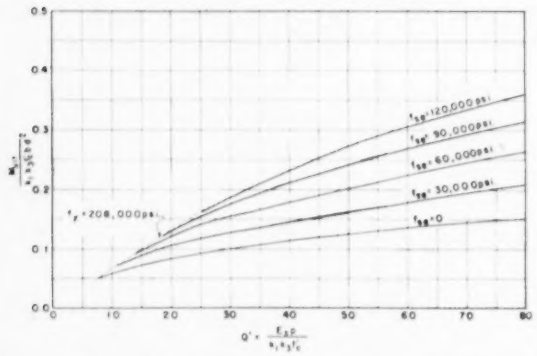


Fig. 8—Computed ultimate moments for beam with Type I steel failing in shear

*Billet, D. F., and Appleton, J. H., "Flexural Strength of Prestressed Concrete Beams," ACI JOURNAL, June 1954, *Proc.* V. 50, pp. 837-854.

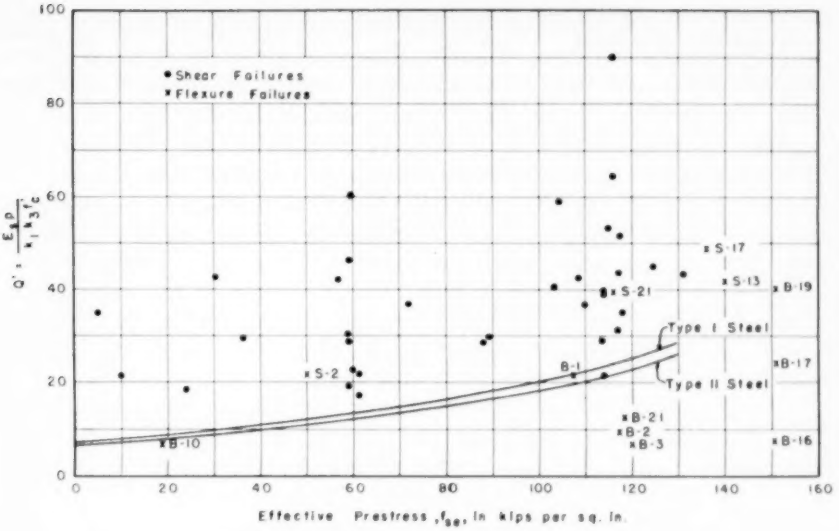


Fig. 9—Effect of Q′ and f_{se} on mode of failure

The data in Fig. 9 are limited to beams of the type considered in these tests. The plotted curves do not appear to apply for effective prestresses greater than 130,000 psi. Above this value, the beams tested developed their ultimate flexural capacities before inclined cracks had formed. It may also be noted that beams S-2 and S-21 failed in flexure although the theory predicted that they should fail in shear. In both of these beams the inclined cracks for some reason did not develop to the extent that they did in the other beams, and the limiting moment computed for shear failure consequently did not govern their behavior. It is evident from these anomalies that additional data are needed before the mode of failure can be predicted with confidence in all cases.

NOMINAL SHEAR STRESSES AND PRINCIPAL TENSION STRESSES

In the present ACI Building Code (ACI 318-51) the design of ordinary reinforced concrete beams in shear is based on the ratio of the nominal shear stress to the cylinder strength of the concrete. The nominal shear stress is computed from the following formula:

$$v = \frac{V}{bjd} \qquad (13)$$

where j is based on a transformed section and calculated by the usual straight line theory.

Nominal shear stresses computed from Eq. (13) for the maximum load on the beam, and the ratio of these stresses to the cylinder strengths of the concrete, are given in Table 2. No use is made of these data in the interpretation of the test results other than to note that in some beams the nominal

ultimate shear stress was very low and that this quantity apparently is not a satisfactory measure of the ultimate strength of the beam. This, of course, is because Eq. (13) involves only the shear whereas the strength of a beam failing in shear seems to be related more closely to the moment.

Current methods used for the design of prestressed concrete beams involve the calculation of principal tension stresses at working loads, presumably as an indication of the behavior of the beam in shear. Values of the maximum principal tension stress S_t have been computed for the beams having the higher prestresses for a "working load" equal to the load at which the stress in the bottom fiber of the beam is zero. Since the beams of this investigation having prestresses less than 120,000 psi were not representative of typical designs, it was not considered desirable to assign "working loads" to these beams on the basis mentioned.

Values of the maximum principal tension stress S_t at working loads are given in Table 2, together with ratio of S_t to f_c'. In all cases the stresses were quite small and seemed to offer no clue as to whether or not the beam would fail in shear or whether web reinforcement should be used.

ULTIMATE LOAD CALCULATIONS FOR ORDINARY REINFORCED CONCRETE BEAMS

Since there seemed to be no reason why the method developed for calculating the ultimate load for beams failing in shear should not apply in principle to beams of ordinary reinforced concrete as well as to prestressed concrete beams, a comparison was made with the results of tests on ordinary reinforced concrete beams. A survey was made of the literature relating to tests of simple beams failing in shear and data were found for 27 beams without web reinforcement and 55 beams with web reinforcement. Test data for these beams are given in Table 3.

In the study of prestressed concrete beams, the two empirical factors, k_1k_3 and $K\epsilon_u$, were evaluated on the basis of two sets of experimental data: the measured depths to the neutral axis k_ud, and the observed moments at failure. For the ordinary concrete beams in Table 3, however, only the observed ultimate moments were available and consequently only one of the two empirical factors could be evaluated on the basis of the available test data. It was assumed, therefore, that Eq. (9) for k_1k_3 could be used also for ordinary reinforced concrete beams, and the value of $K\epsilon_u$ giving the best agreement between test and theory was determined by plotting $M_{ult}/k_1k_3f_c'bd^2$ vs. Q' for various values of $K\epsilon_u$. This procedure gave a value of $K\epsilon_u = 0.00020$ for beams without web reinforcement which is significantly different from the value of 0.00040 that would be obtained from Eq. (11) for prestressed beams with $f_{se} = 0$. It is possible that a satisfactory empirical relation could have been derived for ordinary reinforced concrete beams by assuming $K\epsilon_u = 0.00040$ and determining an expression for k_1k_3 different from that given by Eq. (9). However, the lack of data regarding the depth to the neutral axis for the ordinary reinforced concrete beams and the incompleteness of our present knowledge of the exact mechanism of shear failure

TABLE 3—RESULTS FROM TESTS OF ORDINARY REINFORCED CONCRETE BEAMS

Beam No.	Concrete strength f'_c, psi	Yield stress f_y, psi	Long. reinf. p	Web reinf. r	Shear span a, in.	$Q' = \frac{E_s p}{k_1 k_2 f'_c}$	$u = \frac{f_{su} p}{k_1 k_2 f'_c}$	Ult. load P_{ult}, lb	$M_{ult}/k_1 k_2 f'_c bd^2$		
									Meas.	Comp.	Meas. / Comp.
Bulletin No. 166, University of Illinois Engineering Experiment Station, Series of 1922 Span = 9 ft. Loads at third points.											
221.1	4076	52400	0.0233	——	36	142	0.154	149400	0.155	0.144	1.076
221.2	3696	52400	0.0233	——	36	153	0.161	148000	0.165	0.150	1.100
222.1	4522	52400	0.0233	——	36	131	0.149	165500	0.159	0.140	1.136
222.2	4337	52400	0.0233	——	36	135	0.150	126000	0.124	0.141	0.879
2210.1	3610	52400	0.0093	——	36	62	0.105	180300	0.082	0.100	0.820
2210.2	3570	52400	0.0093	——	36	63	0.106	167200	0.077	0.101	0.762
Moretto, O., "An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams," ACI JOURNAL, Nov. 1945, *Proc.* V. 42, pp. 141-164. Span = 8 ft. Loads at third points. Compression reinforcement not taken into account.											
1-V¼	3460	55000	0.040	0.0028	32	277	0.252	116600	0.235	0.233	1.009
2-B¼	4805	55000	0.040	0.0028	32	216	0.237	135100	0.212	0.213	0.995
1-I¼	3795	55000	0.040	0.0028	32	257	0.257	120950	0.226	0.229	0.987
2-I¼	4700	55000	0.040	0.0028	32	219	0.239	132000	0.207	0.215	0.963
1-D¾	3475	55000	0.040	0.0028	32	275	0.263	118550	0.238	0.234	1.017
2-D¼	3455	55000	0.040	0.0028	32	277	0.264	140550	0.283	0.235	1.204
1-V⅜	3115	47900	0.040	0.0062	32	300	0.323	146850	0.321	0.279	1.151
2-V⅜	4120	47900	0.040	0.0062	32	242	0.294	143650	0.253	0.258	0.981
1-I⅜	3575	47900	0.040	0.0062	32	269	0.309	159000	0.311	0.269	1.156
2-I⅜	4185	47900	0.040	0.0062	32	239	0.293	168000	0.292	0.257	1.136
1-D⅜	2855	47900	0.040	0.0062	32	324	0.334	129750	0.306	0.287	1.066
2-D⅜	3685	47900	0.040	0.0062	32	263	0.305	143650	0.275	0.267	1.030
1-V½	3665	50700	0.040	0.0112	32	264	0.360	157000	0.301	0.306	0.984
2-V½	4815	50700	0.040	0.0112	32	215	0.330	186400	0.292	0.284	1.028
1-I½	3090	50700	0.040	0.0112	32	303	0.378	177750	0.392	0.318	1.233
2-I½	4210	50700	0.040	0.0112	32	238	0.344	196200	0.339	0.294	1.153
1-D½	2705	50700	0.040	0.0112	32	338	0.394	155000	0.381	0.329	1.158
2-D½	3495	50700	0.040	0.0112	32	274	0.354	175825	0.351	0.308	1.140
1a-V¼	3540	46000	0.0188	0.0028	32	128	0.185	106100	0.210	0.171	1.228
"Behavior of Structural Elements under Impulsive Loads—III," Massachusetts Institute of Technology, July 1951. Span = 5 ft for all beams except T-2 for which it was 4 ft. Loaded at midspan. Equal tension and compression reinforcement; compression reinforcement not taken into account.											
T-2	3580	——	0.0140	——	24	95	0.126	10000	0.138	0.119	1.160
T-3	3470	——	0.0314	——	30	217	0.188	8670	0.152	0.173	0.879
T-5	3460	——	0.0218	——	30	151	0.158	9870	0.174	0.148	1.176
T-6	3130	——	0.0140	——	30	104	0.132	8000	0.152	0.125	1.216
T-11	4190	——	0.0140	——	30	84	0.120	12000	0.117	0.114	1.026
T-12	4880	——	0.0218	——	30	117	0.141	15000	0.142	0.133	1.068
Clark, A. P., "Diagonal Tension in Reinforced Concrete Beams," ACI JOURNAL, Oct. 1951, *Proc.* V. 48, pp. 145-156. Span varied from 8-10 ft. Loaded with two symmetrically placed concentrated loads.											
AO-1	3120	53700	0.0098	——	36	74	0.113	40000	0.095	0.108	0.880
AO-2	3770	53700	0.0098	——	36	63	0.106	48500	0.099	0.101	0.980
BO-1	3420	53700	0.0098	——	30	68	0.109	54400	0.100	0.104	0.962
BO-2	3468	53700	0.0098	——	30	68	0.108	42400	0.077	0.103	0.748
BO-3	3410	53700	0.0098	——	30	68	0.109	57600	0.107	0.104	1.029
CO-1	3580	53700	0.0098	——	24	66	0.108	78400	0.112	0.103	1.087
CO-3	3420	53700	0.0098	——	24	68	0.109	75100	0.111	0.104	1.067
DO-1	3750	53700	0.0098	——	18	64	0.105	99600	0.102	0.101	1.010
DO-3	3765	53700	0.0098	——	18	63	0.106	100400	0.103	0.101	1.020
A1-1	3575	46500	0.0310	0.0038	36	209	0.230	100032	0.214	0.208	1.029
A1-2	3430	46500	0.0310	0.0038	36	215	0.232	94032	0.207	0.209	0.990
A1-3	3395	46500	0.0310	0.0038	36	218	0.232	100032	0.223	0.209	1.067
A1-4	3590	46500	0.0310	0.0038	36	208	0.232	110032	0.234	0.209	1.120
B1-1	3388	46500	0.0310	0.0037	30	218	0.233	125370	0.234	0.210	1.114
B1-2	3680	46500	0.0310	0.0037	30	204	0.236	115370	0.201	0.213	0.944
B1-3	3435	46500	0.0310	0.0037	30	215	0.233	128061	0.235	0.210	1.119
B1-4	3380	46500	0.0310	0.0037	30	218	0.234	120561	0.224	0.211	1.062
B1-5	3570	46500	0.0310	0.0037	30	209	0.229	108561	0.194	0.207	0.937
B2-1	3370	46500	0.0310	0.0073	30	218	0.296	135370	0.253	0.259	0.977
B2-2	3820	46500	0.0310	0.0073	30	197	0.287	144870	0.245	0.252	0.972
B2-3	3615	45600	0.0310	0.0073	30	205	0.293	150561	0.266	0.257	1.035
B6-1	6110	46500	0.0310	0.0037	30	143	0.210	170561	0.208	0.192	1.083
C1-1	3720	46500	0.0207	0.0034	24	135	0.200	124870	0.172	0.183	0.940
C1-2	3820	46500	0.0207	0.0034	24	132	0.198	139870	0.189	0.182	1.038
C1-3	3475	46500	0.0207	0.0034	24	142	0.204	110561	0.161	0.186	0.866
C1-4	4210	46500	0.0207	0.0034	24	123	0.191	128561	0.162	0.176	0.920

Continued on p. 196

Beam No.	Concrete strength f'_c, psi	Yield stress f_s, psi	Long. reinf. p	Web reinf. r	Shear span a, in.	$Q' = E_s p / k_1 k_3 f'_c$	$k_u = f_{su} p / k_1 k_3 f'_c$	Ult. load P_{ult}, lb	$M_{ult}/k_1 k_3 f'_c bd^2$		
									Meas.	Comp.	Meas./Comp.
C3-1	2040	46500	0.0207	0.0034	24	223	0.251	109561	0.229	0.225	1.018
C3-2	2000	46500	0.0207	0.0034	24	226	0.250	99051	0.208	0.224	0.929
C3-3	2020	46500	0.0207	0.0034	24	224	0.248	84561	0.194	0.222	0.874
C4-1	3550	46500	0.0310	0.0034	24	210	0.243	139061	0.200	0.218	0.917
C6-2	6560	46500	0.0310	0.0034	24	137	0.203	190561	0.179	0.186	0.962
C6-3	6480	46500	0.0310	0.0034	24	139	0.203	195561	0.185	0.186	0.995
C6-4	6900	46500	0.0310	0.0034	24	134	0.200	192717	0.176	0.183	0.962
D1-6	4010	46500	0.0342	0.0046	24	210	0.259	78561	0.210	0.231	0.909
D1-7	4030	46500	0.0342	0.0046	24	208	0.261	80561	0.214	0.232	0.922
D1-8	4030	46500	0.0342	0.0046	24	210	0.261	83561	0.223	0.232	0.961
E1-2	4375	46500	0.0342	0.0073	25	197	0.284	99717	0.251	0.250	1.044
D2-6	4280	46500	0.0342	0.0061	30	201	0.274	75717	0.242	0.242	1.000
D2-7	4120	46500	0.0342	0.0061	30	206	0.277	70717	0.232	0.245	0.947
D2-8	3790	46500	0.0342	0.0061	30	220	0.282	75717	0.265	0.249	1.068
D4-1	3970	46500	0.0342	0.0049	30	213	0.287	75717	0.257	0.253	1.016
D4-2	3720	46500	0.0342	0.0049	30	223	0.294	70717	0.251	0.259	0.969
D5-1	4020	46500	0.0342	0.0037	30	210	0.247	65717	0.220	0.221	0.996
D5-2	4210	46500	0.0342	0.0037	30	203	0.244	70717	0.223	0.219	1.046
D5-3	3930	46500	0.0342	0.0037	30	214	0.248	70717	0.241	0.222	1.086

Laupa, A., Siess, C. P., and Newmark, N. M., "Shear Strength of Simple-Span Reinforced Concrete Beams Without Web Reinforcement," *Structural Research Series* No. 52, Civil Engineering Studies, University of Illinois, 1953. Span = 9 ft. Loaded through a column stub at midspan.

S-2	3900	45000	0.0208	——	48	131	0.148	19092	0.143	0.139	1.029
S-3	4689	45000	0.0252	——	48	139	0.152	23882	0.160	0.142	1.127
S-4	4470	45000	0.0321	——	48	183	0.176	24992	0.177	0.163	1.086
S-5	4331	45000	0.0411	——	48	238	0.198	22390	0.163	0.182	0.896
S-11	2140	47500	0.0190	——	48	198	0.182	15200	0.191	0.168	1.137
S-13	3800	44100	0.0411	——	48	264	0.207	22390	0.180	0.189	0.952

Average = 1.021
Standard Deviation = 0.10

makes it difficult to choose between the various ways of deriving empirical expressions for the shear strength of ordinary beams. The procedure used herein is only one possible way of attacking this problem.

A similar study of the beams with web reinforcement led to the following expression for the parameter $K\epsilon_u$:

$$K\epsilon_u = 0.00020 + 0.05r \quad (14)$$

where $r = A_v/sb$ = ratio of web reinforcement
A_v = area of two single stirrups at a cross section
s = stirrup spacing, measured normal to the direction of the stirrups
b = width of the beam

The values of $K\epsilon_u$ determined by the empirical method described are plotted as a function of the ratio of web reinforcement r in Fig. 10. No shear failures were found in simple beams having ratios of web reinforcement greater than about 1.2 percent. For this reason Eq. (14) is not valid for r greater than this value. Beams with ratios of web reinforcement greater than 1.2 percent have been tested, but all of those which were considered in this study resulted in flexure failures.

The similarity between Eq. (11) and (14) suggests that web reinforcement in ordinary reinforced concrete beams and prestress in prestressed concrete beams have similar effects on the mode of failure of simple beams. As either the web reinforcement or the prestress is increased, the depth to the neutral axis at ultimate load, and thus the ultimate load, is increased for a beam with given properties. The increase in depth to the neutral axis is brought about by the restraint against development of inclined cracking which the

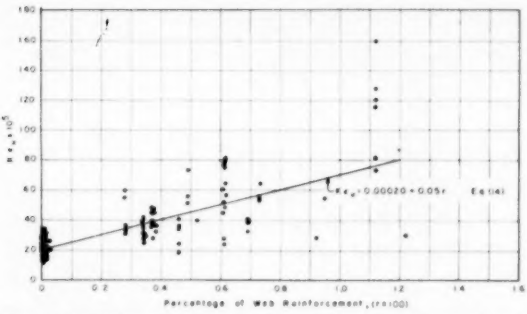


Fig. 10—Relation between $K\epsilon_u$ and percentage of web reinforcement for ordinary reinforced concrete beams

presence of web reinforcement and prestress impose on the beam. Although web reinforcement does not delay the formation of diagonal tension cracks as does the presence of prestress, it does retard their development once they have formed, and tends to cause a better distribution of cracks in the shear region of the beam.

Substitution of Eq. (14) into the expressions previously developed for prestressed concrete beams, but with the prestress equal to zero yields the relations between $M_{ult}/k_1k_3f_c'bd^2$ and Q' shown in Fig. 11. The form of this relation for beams failing in shear is seen to be similar to that for beams failing in compression.

The division between shear and tension failure can be assumed to correspond to an ultimate steel stress equal to the yield point. If this condition is substituted into Eq. (7), together with $K\epsilon_u$ from Eq. (14), the following expression is obtained for the ratio of web reinforcement necessary to prevent shear failure in a beam having a particular value of Q'.

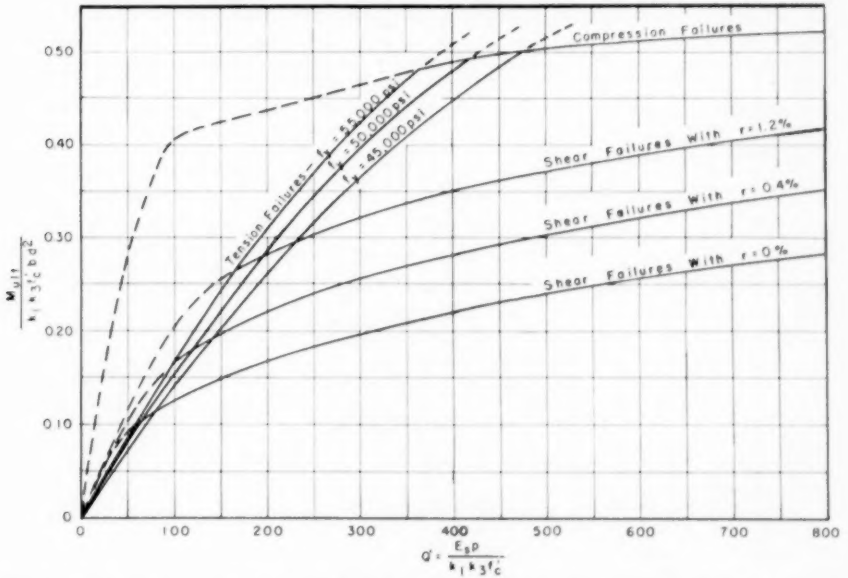

Compression Failures
Tension Failures — f_s = 55,000 psi
f_s = 50,000 psi
f_s = 45,000 psi
Shear Failures With r = 1.2%
Shear Failures With r = 0.4%
Shear Failures With r = 0%
$\frac{M_{ult}}{k_1k_3f_c'bd^2}$
$Q' = \frac{E_sp}{k_1k_3f_c'}$

Fig. 11—Relation between M_{ult} and Q' for tension, shear, and compression failures in ordinary reinforced concrete beams

$$r = \frac{\frac{Q'f_y}{E_s}\left(\frac{f_y}{E_s} + 0.00020\right) - 0.00020}{0.05\left(1 - \frac{Q'f_y}{E_s}\right)} \quad \ldots\ldots (15)$$

Eq. (15) has been derived only for simple beams subjected to one or two symmetrically placed loads, and is not valid for values of r greater than about 1.2 percent since no shear failures occurred above this value for the beams studied. For this same reason, it is not possible from available data to determine the boundary between shear and compression failures. It seems evident from the curves in Fig. 11 that simple beams cannot be made to fail in compression unless a fairly large amount of web reinforcement is provided, or unless the ratio of moment to shear is large enough that the flexural mode of failure dominates and inclined cracks do not have a chance to develop. None of the test data studied contradicted this conclusion.

The test results for ordinary reinforced concrete beams failing in shear that were studied in connection with the development of Eq. (14) are given in Table 3. Measured and computed ultimate moments are compared in the table and also in Fig. 12. The values of f_{su} used in the calculations and in computing k_u for Fig. 12 were obtained from Eq. (7) modified for zero prestress and for the stress-strain characteristics of ordinary reinforcement. The following equation was used:

$$f_{su} = (K\epsilon_u\, k_1k_3f_c'/p)/\left(\frac{f_{su}}{E_s} + K\epsilon_u\right) \quad \ldots\ldots (16)$$

The term k_1k_3 and $K\epsilon_u$ were determined from the empirical Eq. (9) and (14), respectively.

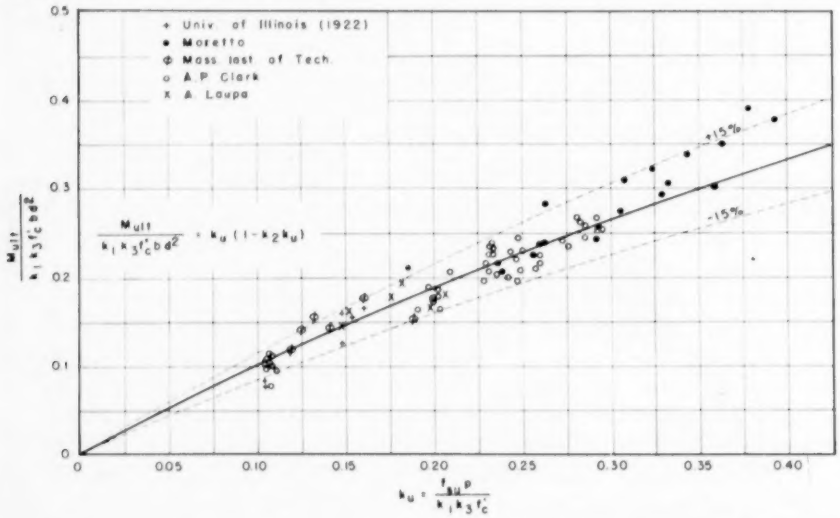


Fig. 12—Comparison of measured and computed ultimate resisting moments for ordinary reinforced concrete beams failing in shear

The agreement between measured and computed ultimate resisting moments of ordinary reinforced concrete beams failing in shear is seen from Table 3 and Fig. 12 to be satisfactory. The average ratio of measured to computed moment was 1.021, with a standard deviation of 0.10.

SUMMARY

The principal conclusions that can be drawn from the tests and analyses presented in this paper are as follows.

1. Ultimate failure in shear of simply-supported rectangular prestressed concrete beams of the type tested occurred by crushing of the concrete in the compression zone of the beam over an inclined crack. Crushing occurred when the strain in the concrete reached a limiting value, and the ultimate load was controlled by moment rather than by shear. The effect of shear was to cause the formation and propagation of inclined cracks which, for shear failures, penetrated a greater distance into the compression zone than vertical cracks formed in a region of pure flexure. Shearing stresses in the compression zone of the beam were not considered to contribute to the failure.

2. Ultimate strength of a beam failing in shear can be expressed by equations of the same type as those used to express ultimate moment capacity of a beam failing in flexural compression. These equations, however, have been modified empirically to take into account the reduced capacity of the compression zone resulting from the greater height of the inclined cracks and the concentration of angle change at the critical section.

3. On the basis of limited studies involving only simply-supported beams, the concepts of shear failure developed for prestressed concrete beams seem to apply equally well to beams of ordinary reinforced concrete.

4. An increase in prestress in the prestressed concrete beams or an increase in the amount of web reinforcement in the ordinary reinforced concrete beams effectively increased the strength in shear. The effect of these two variables is believed to be similar since both act either to delay or inhibit the formation and propagation of inclined cracks.

The conclusions stated above are of course limited in applicability by the scope of the tests. It should be mentioned, therefore, that the investigation of shear strength of prestressed concrete beams is being continued and that additional variables are being studied. Special attention is being given to the strength and behavior of I-beams.

ACKNOWLEDGMENT

This study was carried out in the Structural Research Laboratory of the University of Illinois as part of a cooperative investigation of prestressed concrete for highway bridges sponsored by the Illinois Division of Highways as part of the Illinois Cooperative Highway Research Program. The U. S. Bureau of Public Roads participated through grants of federal aid funds. This paper is based on a Ph. D. thesis prepared by E. M. Zwoyer in the

Department of Civil Engineering under the direction of C. P. Siess and N. M. Newmark, research professor of structural engineering.

Thanks are extended to P. C. Gardner, Research Assistant in Civil Engineering, for his valued assistance in connection with the tests and analyses, and to J. H. Appleton, research associate in civil engineering, for his help in solving many problems associated with the test program and for his assistance in all phases of the work and in the preparation of this paper.

NOTATION

Wherever possible, the notation used is consistent with that recommended by the Joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete.*

a = shear span, distance between load point and nearest support

A_s = total area of tension reinforcement

A_v = area of web reinforcement in ordinary reinforced concrete beams

b = width of rectangular section

C = total internal compressive force in concrete at ultimate load (Fig. 4)

d = effective depth to center of gravity of reinforcement

E_s = modulus of elasticity of steel

f_c' = compressive strength of 6 x 12-in. cylinders

f_{se} = steel stress due to effective prestress force at time of testing

f_{su} = maximum steel stress at failure of beam

f_y = yield stress of mild steel reinforcement, or 0.2 percent offset stress of wire reinforcement in prestressed beams

j = ratio of lever arm of tensile reinforcement to effective depth to steel computed by straight-line theory for ordinary reinforced concrete beams

k_1 = ratio of area under concrete compressive stress block at ultimate load to area of enclosing rectangle

k_2 = ratio of distance from top of beam to centroid of the compressive stress block to depth of the neutral axis (Fig. 4)

k_3 = ratio of maximum concrete compressive stress in beam to compressive strength of standard 6 x 12-in. cylinders (Fig. 4)

$k_u d$ = distance from top of beam to neutral axis at ultimate load (Fig. 4)

$K_1 \epsilon_u$ = average compressive strain on top surface of the beam over some length of the beam at ultimate load

$K_2 \epsilon_{cu}$ = average concrete strain at the level of the reinforcement over some length of the beam at ultimate load

K = K_1/K_2 = an empirically determined parameter

L = span length of the beam

M_{ult} = bending moment at ultimate load

p = A_s/bd = ratio of tension reinforcement

P = total concentrated load on beam

P_{ult} = total concentrated load at failure

Q' = $E_s p/k_1 k_2 f_c'$ = a parameter defining the properties of a beam

r = A_v/sb = ratio of web reinforcement in ordinary reinforced concrete beams

s = spacing of stirrups, measured normal to the direction of the stirrups

S_t = principal tensile stress

T = total tensile force in reinforcement at ultimate load (Fig. 4)

v = V/bjd = nominal unit shearing stress

V = shear in beam

V_{ult} = shear at ultimate load

ϵ_{co} = compressive strain in concrete at the level of the reinforcement due to effective prestress force

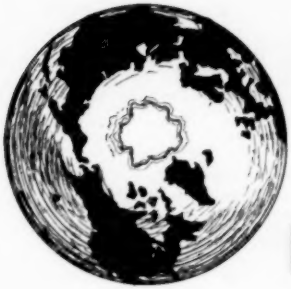
ϵ_{cu} = strain in concrete at the level of the reinforcement at the location of maximum moment at ultimate load

ϵ_{se} = strain in steel due to effective prestress force

ϵ_{su} = strain in steel at the location of maximum moment at ultimate load

ϵ_u = ultimate compressive strain in concrete

*ACI-ASCE Committee 323, "Proposed Definitions and Notations for Prestressed Concrete," ACI JOURNAL, Oct. 1952, *Proc.* V. 49, pp. 85-88.



Letters from Readers*

BY WAY OF SYNOPSIS

By using the Maclaurin series to represent the elastic curve, K. T. Sundara Raja Iyengar presents a direct method for determining fixed-end moments in beams with irregular loading.

Irving B. Rau describes the use of concrete ring wales in simplifying cofferdam construction.

D. G. Norman presents references of published information on a unit water content law for mix design.

Determination of Fixed-End Moments in Beams with Irregular Loading Without Use of Formulas from Handbooks (LR 51-6)†

The method of determining fixed-end moments in beams with irregular loading without use of formulas from handbooks as presented by Joseph Zalkin[1] makes use of false reactions. Another method suggested by L. Rudchenko and W. E. McNely[2] divides triangular loads into 1-ft segments, and the incremental loads are considered as concentrated loads at the center of each segment; moments due to the individual concentrated loads are superimposed and the fixed-end moments are obtained.

A more direct solution to this problem is presented here by representing the elastic line of the beam by the Maclaurin series (a method due to Hetényi[3]). This method is explained here and the same numerical example used by Zalkin,[1] and Rudchenko and McNely,[2] is worked out by this method for comparison.

Elementary theory of bending of beams, based on the assumption that plane cross sections of a beam remain plane after deformation, gives the following equation for the bending moment at a point in a beam[4]

$$EI \frac{d^2y}{dx^2} = -M \quad \ldots\ldots (1)$$

By successive differentiation, we obtain

$$EI \frac{d^3y}{dx^3} = -V \quad \ldots\ldots (2)$$

$$EI \frac{d^4y}{dx^4} = q \quad \ldots\ldots (3)$$

where M = bending moment at any point
V = shear at the point
q = intensity of the distributed loading
EI = flexural rigidity

If $y = f(x)$ represents the deflection curve (Fig. 1a) for the beam, the value of y at any point x from the origin is given by the Maclaurin series:

$$y = f(0) + \frac{x}{1!} f^{I}(0) + \frac{x^2}{2!} f^{II}(0) +$$

*A part of copyrighted Journal of the American Concrete Institute, V. 26, No. 2, Oct. 1954, *Proceedings* V. 51. Separate prints of the entire Letters from Readers section are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†See also ACI Journal, Oct. 1951, *Proc.* V. 48, p. 186; Jan. 1952, *Proc.* V. 48, p. 425.

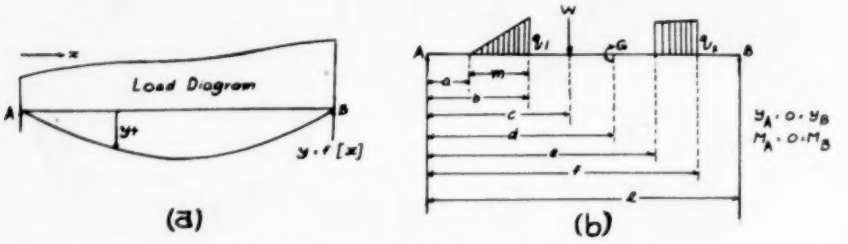


Fig. 1a—Deflection curve for $y = f(x)$. Fig. 1b—General case for Eq. (4) as an extension of Macaulay's method

$$\frac{x^3}{3!} f'''(0) + \ldots + \frac{x^n}{n!} f^n(0) \ldots\ldots\ldots\ldots (4)$$

Where $f(0), f'(0), \ldots f^n(0)$ denote the values of $f(x)$ and its derivitives at the origin, $x = 0$. Thus we have

$$\begin{aligned} f(0) &= y_o \text{ (deflection at the origin)} \\ f'(0) &= \theta_o \text{ (slope at the origin)} \\ f''(0) &= -M_o/EI \\ f'''(0) &= -V_o/EI \\ f^{IV}(0) &= q_o/EI \end{aligned} \quad \ldots\ldots (5)$$

The advantage of using this equation for the elastic curve lies in the fact that generally the physical nature of the problem gives the values of y_o, θ_o, M_o, V_o, and q_o easily. For example, in a simply supported beam, with the left support taken as the origin, we have $y_o = 0$, $y_{x=l} = 0$, $M_o = 0$, and $M_{x=l} = 0$. V_o and $V_{x=l}$ are known from statics. Hence the series for y can be written easily and any quantity (moment, shear, slope, or deflection) at any point along the beam, can be calculated by simple substitution. This method is equally applicable to statically indeterminate beams, as will be shown in the numerical example which follows.

Representing the elastic curve $y = f(x)$ by the Maclaurin series assumes continuity of the function representing the deflection curve and all its derivatives. However, in the case of a concentrated loading, a discontinuity occurs in the shear force at the point of application of the load, *i.e.*, a discontinuity in the third derivative of $y = f(x)$. Similarly, a couple acting at a point produces a discontinuity in the bending moment at the point, *i.e.*, in the second derivative of the function. To cover all such cases, Eq. (4) can be modified as an extension of Macaulay's method.

Referring to Fig. 1b the modified equation will be

$$y = \theta_o x - \frac{1}{3!}\frac{V_o x^3}{EI} + \frac{1}{5!}\frac{q_1 (x-a)^5}{m E I} - \frac{1}{4!}\frac{q_1 (x-b)^4}{E I} - \frac{1}{5!}\frac{q_1 (x-b)^5}{m E I} + \frac{1}{3!}\frac{W (x-c)^3}{E I} - \frac{1}{2!}\frac{G (x-d)^2}{E I} + \frac{1}{4!}\frac{q_2 (x-e)^4}{E I} - \frac{1}{4!}\frac{q_2 (x-f)^4}{E I} \ldots\ldots\ldots (6)$$

The terms in the brackets should be interpreted as (where z is any function)

$(z)^n = z^n$ if z is positive

$(z)^n = 0$ if z is negative

With such modifications, the method is applicable to any system of arbitrary loading.

Example

In the numerical example in Fig. 2 $\theta_A = \theta_B = 0$, and $y_A = y_B = 0$. The series for the deflection curve for the fixed beam is (taking A as origin)

$$y = -\frac{1}{2!}\frac{M_A x^2}{E I} - \frac{1}{3!}\frac{V_A x^3}{E I} + \frac{1}{5!}\frac{2 (x-5)^5}{3 E I} - \frac{1}{5!}\frac{2 (x-8)^5}{3 E I} - \frac{1}{5!}\frac{2 (x-8)^5}{4 E I} + \frac{1}{5!}\frac{2 (x-12)^5}{4 E I} \ldots\ldots\ldots (7)$$

Putting $x = 15$, we have

$0 = -112.5 M_A - 562.5 V_A + 393.17 \ldots (8)$

Differentiating Eq. (7) and putting $x = 15$

$0 = -15 M_A - 112.5 V_A + 162.76 \ldots\ldots (9)$

Eliminating V_A from Eq. (8) and (9)

$$M_A = -11.22 \text{ kip-ft}$$

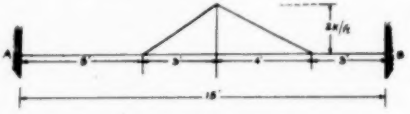


Fig. 2—Beam for numerical example

Differentiating Eq. (7) twice and using Eq. (1)

$$Mx = M_A + V_A x - \frac{1}{3!}\frac{2(x-5)^3}{3EI} + \frac{1}{3!}\frac{2(x-8)^3}{3EI} + \frac{1}{3!}\frac{2(x-8)^3}{4EI} - \frac{1}{3!}\frac{2(x-12)^3}{4EI} \quad \ldots\ldots (10)$$

Substituting $x = 15$ and $M_A = -11.22$ kip-ft

$M_B = -11.22 + 15V_A - 46.67$ ..... (11)

Eliminating V_A from Eq. (9) and (11) yields

$M_B = -13.75$ kip-ft

References

1. Zalkin, Joseph, "Determination of Fixed End Moments in Beams with Irregular Loading Without Use of Formulas from Handbooks (LR 48-12)," ACI JOURNAL, Oct. 1951, *Proc.* V. 48, p. 186.

2. Rudchenko, L., and McNely, W. E., "Determination of Fixed End Moments in Beams with Irregular Loading Without Use of Formulas from Handbooks (LR 48-12)," ACI JOURNAL, Jan. 1952, *Proc.* V. 48, p. 425.

3. Hetényi, M., *Journal of the Franklin Institute*, Nov. 1952, V. 254, No. 5, p. 369.

4. Timoshenko, S., *Strength of Materials*, V. 1, 1951, p. 135.

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Concrete Ring Wales Simplify Cofferdam Construction (LR 51-7)

The New Orleans area was recently introduced to the use of concrete as a structural member in cofferdam construction in the construction of Sewerage Pumping Station No. 22 for the Sewerage and Water Board of New Orleans. This project was located within a mile of Lake Pontchatrain in an area of saturated fine grain sand, much different than the pleistocene clays upon which most of New Orleans is founded. Because of the limited area of the site and the close proximity of adjacent structures an open excavation with sloping sides was not possible. It was therefore decided to use a braced sheet pile cofferdam utilizing circular concrete ring wales (Fig. 1).

Steel sheet piling (M-116) 30 ft long were driven on a 40 ft diameter circle, excavation was commenced, and the concrete ring wales cast directly on the soil when the proper elevation was reached. Dewatering was accomplished by a combination of open pumping and wellpoints; the wellpoint pump and header were installed on the second concrete ring wale. Due to the dewatering of the bottom, no tremie concrete was required; only a 6-in. slab of dry mix concrete was specified for a working surface. The total depth of the excavation was 22 ft.

Because the water table in New Orleans is generally close to the surface of the ground, it was assumed to be at the surface, a common design practice in New Orleans that is usually not too conservative. Drawdown due to the wellpoint system was not complete at times because a differential head could exist during installation and when the wellpoint pump was not working. A straight-line equivalent-liquid pressure diagram was used for the soil plus the water, and a constant pressure diagram was used for a surcharge load from a ¾-cu yd crawler crane.

The stress in a circular concrete ring wale with uniform load is theoretically compressive only, but bending moment could have been introduced by any combination of the following:

1. Surcharge loads caused by moving construction equipment

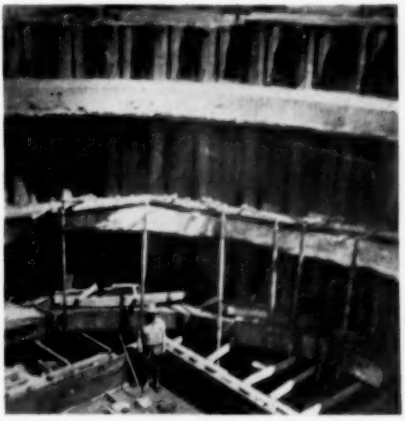

Fig. 1—Braced sheet pile cofferdam utilizing concrete ring wales

2. Surcharge loads caused by stockpiles of materials or of soil from the excavation
3. Soil loads not the same around the circumference due to sloping terrain
4. Soil loads not the same around the circumference due to removal of some of the soil by trenches dug to the cofferdam for the installation of utilities
5. Excavation within the cofferdam not being level
6. Sheet piling and concrete not constructed to a perfect circle

The three concrete ring wales were made identical because an exact analysis of the bending moment was impossible, because it has been found in practice that the pressures on the top portion of bracing are more than the pressures computed on the theory of a straight-line equivalent-liquid pressure distribution, and because of simplicity in the forms and reinforcing steel. Minimum concrete strength was specified to be 3000 psi. The dimensions of the concrete wale were 24 in. vertically and 22 in. horizontally. The horizontal dimension was kept reasonably small to keep from increasing the diameter of the cofferdam. Reinforcing was seven #10 bars in each vertical face held in position by closed hoop stirrups. The concrete ring wales were prevented from moving downward by hangers made of reinforcing bars. One end was embedded in the concrete and the other end was welded to the sheet pile. These hangers were on 5 ft 4 in. centers.

Upon completion of the pumping station substructure backfill was commenced. When the backfill reached the bottom of the concrete ring wales the embedded hangers were burned off and the concrete broken by jackhammers into large pieces and disposed of.

By using this system of bracing an unobstructed area was obtained. This allowed the excavation, foundation pile driving, reinforced concrete construction, and backfill to proceed unencumbered by the usual cross bracing. The cost of the system as used was cheaper than the cost of the conventional cross bracing, plus the unevaluated savings resulting from a clear work area.

General contractors for the work were R. P. Farnsworth & Co., Inc., under the direction of Pratt Farnsworth, Sr., general superintendent. Cofferdam design was by the author.

IRVING B. RAU, Engineer, R. P. Farnsworth & Co., Inc., New Orleans, La.

Unit Water Content Law for Mix Design (LR 51-8)

Following Mr. Sowers' letter in the June 1953 JOURNAL* there has been some discussion† on the possible existence of a "unit water content law."

I should like to point out that this law was published by Willis A. Slater and Inge Lyse in their paper "Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams," which appeared in the ACI JOURNAL, June 1930, *Proc.* V. 26, pp. 831-874. The relevant chart appears on page 841.

I have confirmed the relationship in my own work and use it as the basis of elementary mix design instruction. Having tried all the known methods of design, I favor (for normal on-the-job mixed concrete) a combination of Abrams' fineness modulus method of grading aggregates (see discussion of "Concrete Mix Design—A Modification of the Fineness Modulus Method," by Stanton Walker and Fred F. Bartel, ACI JOURNAL, Dec. 1947, *Proc.* V. 43, pp. 844-1 844-10) with the Slater-Lyse law, applied in the form of a trial mix to find the unit water content. Cement content follows easily from this and any further adjustments of the proportions would be made purely in the light of subsequent experience with the mix, and should only be of a minor nature.

D. G. NORMAN, Department of Civil Engineering, University of Natal, Durban, South Africa

*Sowers, George B., "Concrete Mix Design (LR 49-22)," ACI JOURNAL, June 1953, *Proc.* V. 49, pp. 959-960.
†Shilstone, Cecil M., "Mix Design (LR 50-16)," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, p. 604.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Dams

Contribution to the analysis of dome-type dams (Contribucion al calculo de diques en cupula)

KARL FEDERHOFER, *Tecnica* (National University of Tucuman, Argentina), V. 1, No. 5, Sept. 1952, pp. 227-233

AUTHOR'S SUMMARY

The method of F. Toelke for the computation of masonry arch dams stems from the differential equation for the radial deflections of a cylindrical tank, the solution of which permits finding the stresses and deformations of such a tank. Modern types of arch dams have curvature not only in the horizontal planes but more or less also in vertical sections.

Author deals with a method to take into consideration the influence of this second curvature on the stresses and deformations found by the first approximation by expanding the above mentioned equation for a cylindrical tank.

Temperature and deformation measurements on a spillway buttress (Temperaturund Dehnungsmessungen an einem Wehrpfeiler)

R. KUHN, *Beton- u. Stahlbetonbau* (Berlin), V. 47, No. 9, Sept. 1952, pp. 213-218; No. 10, Oct. 1952, pp. 234-241

Reviewed by RUDOLPH FISCHL

A detailed report of measurements taken over a period of 2½ years on a buttress of a spillway at Limbach, Germany. Three different kinds of gages were built in the structure to measure deformations, temperatures, and stresses. Results of the measurements are discussed and illustrated with graphs, but no conclusions are drawn for construction or design of this kind of structure.

Use of a gelatin model in the photoelastic analysis of a gravity dam

J. S. ELLIS and K. W. GENT, *The Engineering Journal* (Montreal), V. 36, No. 11, Nov. 1953, pp. 1435-1439

Reviewed by ARON L. MIRSKY

Discusses method used in making a large (22 in. long, 11.5 in. high, and 5 in. thick) gelatin model loaded by pneumatic cushions, and the optical system designed to accommodate it. White light and color photographs made it easy to distinguish between isoclinics and isochromatics; hence the setup was not only simplified by the omission of the quarter-wave plates, but the entire model could be photographed on one film, reducing the number of films and, more importantly, the time required, a distinct advantage since gelatin models have relatively short useful lives.

Design

Relaxation methods: A retrospect

RICHARD SOUTHWELL, *Engineer* (London), V. 196, No. 5105, Nov. 27, 1953, pp. 706-709

Reviewed by ARON L. MIRSKY

Engineers who have used relaxation methods, and those few unfortunates who have merely heard of them, will enjoy this presentation of the place of these methods in engineering. Those who have become a bit worried over the possibility of technologic unemployment due to the increasing number of electronic "brains" will find comfort in author's demonstration that engineers with human brains will still be needed.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 2, Oct. 1954, *Proceedings* V. 51. Address 18263 W. McNichols Rd., Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parenthesis following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

Article is abstract of the 40th Thomas Hawksley Lecture of the Institution of Mechanical Engineers (London, Nov. 20, 1953). The complete lecture is published in *The Chartered Mechanical Engineer* (London), V. 1, No. 2, Feb. 1954, pp. 61-72.

Analysis of rigid-jointed plane frameworks. Use of an electronic digital computor

R. K. Livesley and T. M. Charlton, *Engineering* (London), V. 177, No. 4595, Feb. 19, 1954, pp. 239-241
Reviewed by Aron L. Mirsky

A previous paper by senior author ("Analysis of Rigid Frames by an Electronic Digital Computer," *Engineering*, V. 176, No. 4569, Aug. 21, 1953, pp. 230-233; No. 4570, Aug. 28, pp. 277-278; see "Current Reviews," ACI Journal, May 1954, p. 807) discussed routines for analyzing plane rigid-jointed frameworks on the University of Manchester computer. Size of frames was limited by storage capacity of computer. This paper describes application of electronic computation to frames of any number of degrees of freedom (of which each joint ordinarily has three), by use of technique of successive reduction. Technique involves dividing frame into two or more subframes having certain joints in common.

Paper summarizes theory and describes the program. Results of the analysis of a transverse frame of a power plant for crane surge loading are given; frame had 16 joints, time required to prepare the data tape was about 5 hr, while computer time, including printing out the calculated displacements and internal forces, was about 15 min.

Further research in reinforced concrete, and its application to ultimate load design

Arthur L. L. Baker, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 2, No. 2, Aug. 1953, pp. 269-310
Reviewed by Aron L. Mirsky

In 1951 Professor Baker ("Recent Research in Reinforced Concrete, and its Application to Design," *Journal*, Institution of Civil Engineers (London), V. 35, No. 4, Feb. 1951, pp. 262-329, including discussion) presented a general "plastic" (ultimate load) theory for the analysis of statically indeterminate reinforced concrete frameworks and the calculation of the ultimate strength of reinforced and prestressed concrete beams. Some data on tests of prestressed beams and cylindrical shells were included.

Current paper presents additional data on the tests, and describes a trial-and-error "plastic-hinge" method of analyzing reinforced concrete frames as many as several hundred times indeterminate. Method involves assuming the location of the plastic hinges and the values of the plastic moments in the frames and adjusting them by trial and error until conditions of zero resultant rotation at the hinges and nonexcessive strains or deflections are attained. Goal is selection of hinge locations, plastic-moment values, and relative stiffnesses of members, "so that the distribution of bending moments prior to failure is economic and so that permissible strains and deflexions under working load are not exceeded." Adjustments of the bending strength are made chiefly by varying the area of reinforcement; concrete cross-sections, in general, remain unchanged from those originally assumed.

Of some interest are two innovations: the device of plotting each bending moment case on a separate card and then computing the resultant rotation at each hinge by integration, which is performed by "playing" these cards against each other—the author has very aptly named this "structural patience" (pp. 282-283); an appendix, by C. Y. Yu (pp. 309-310) on the use of the abacus is included—it seems that Mr. Yu demonstrated that the calculations described in the paper could be carried out more quickly with an abacus than with a slide rule or a desk calculating machine.

Additional, and very interesting, discussion appears in the proceedings, Part III, V. 3, No. 1, Apr. 1954, pp. 289-310.

Use of graphs and nomographs in engineering design (in Spanish)

Eduardo Arnal, *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 213, Dec. 1953, pp. 21-31; No. 214, Jan. 1954, pp. 4-12
Reviewed by Joseph J. Waddell

This article, in two parts, covers the construction of nomographs and charts for the graphical solution of problems. In the words of the author: "The ideas expounded in these articles are not claimed to cover the extensive field of modern nomography; their principal object is to awaken (in the reader) curiosity concerning this valuable tool for the designer."

While there is abundant literature on this subject, most of it is calculated in English

units, or is based on standards and coefficients different from those commonly accepted in Venezuela (or in other countries of Central and South America). Hence this article should be of special interest to all Spanish-speaking engineers.

Starting with a brief history, definitions, and table of notation, the article covers the construction of nomographs by means of text, illustrative examples, and numerous figures. Because the usual systems for the graphical representation of the different variables of any equation are numerous, five general keys for the construction of nomographs are presented under which virtually all of the systems can be included.

Determination of the stresses in a beam of prismatic section (Determinacion de las tensiones en el timpano de sistemas plegados)

HERMANN BEER and GENARO R. A. PISCITELLI, *Tecnica* (National University of Tucuman, Argentina), V. 1, No. 6, Dec. 1952, pp. 325-335

AUTHORS' SUMMARY

In continuation of the article published in *Tecnica*, V. 1, No. 3, 1951, there is treated here the problem of the nonrectangular section, exemplified by the cross section of a spandrel with a nonrectangular section.

Authors apply the external stress function, dealt with in the above mentioned article; analysis is by the method of finite differences in rhomboidal nets.

Pavements

Report of committee on curing of concrete pavements

J. H. SWANBERG, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 335-342

AUTHOR'S SUMMARY

This report is a summary of replies to a questionnaire prepared by the Committee on Curing of Concrete Pavements of the Highway Research Board and sent to each state and to five federal agencies. Information was received on such items as (1) curing methods permitted and extent of their use, (2) test methods, acceptance limits, kinds, rate of application and preferences of membrane-forming compounds, (3) provisions governing the opening of pavements, (4) order of preference of curing methods or materials, and (5) special conditions provided for curing concrete in cold weather.

The data indicate that the most popular methods on the basis of extent of use are membrane-forming compounds, waterproof paper, and wet burlap, followed by earth, hay, or straw. The preference of clear and pigmented membrane compounds is about equally divided. The rate of application of membrane compounds showed considerable variation with about half requiring a rate of not more than 200 sq ft per gal. of compound. The test methods and acceptance limits of membrane compounds varied considerably.

There was not much agreement as to preference of curing methods or materials. A substantial difference was found in the provisions governing the opening of pavements both from the standpoint of strength or age of the pavement. There was close agreement on the requirements for curing of concrete in cold weather.

Soil cement and concrete roads

Engineering (London), V. 176, No. 4583, 4584, Nov. 27, 1953, pp. 681-682; Dec. 4, 1953, p. 724

Reviewed by ARON L. MIRSKY

Summaries of six papers, on various aspects of construction of soil-cement and concrete roads, delivered at the annual meeting of the Pavings Development Group of the Cement and Concrete Assn. in London, Nov. 1953 (one of the papers was reviewed in "Current Reviews," ACI JOURNAL, Sept. 1954, *Proc.* V. 51, p. 112).

Precast concrete

Vermiculite concrete for roofs and wall panels

L. A. CASTELL, *Rock Products*, V. 56, No. 6, June 1953, pp. 196, 197

Describes precast concrete joists, precast vermiculite concrete wall panels, and construction procedure for cast-in-place vermiculite concrete roof slab.

Deep piers formed with huge precast units

Engineering News-Record, V. 152, No. 9, Mar. 4, 1954, pp. 30-32, 35

Reviewed by S. J. CHAMBERLIN

Deep-water piers for a 4-mile bridge across San Francisco Bay are being built of precast concrete shells assembled under water to serve as outside forms for tremie-placed concrete. Steel piles up to 200 ft long are driven through a heavily reinforced concrete template which is also used as the bottom form for the tremie concrete and as a seat for the lower shell sections. Space between the pile and slots in the grid are tremie grouted to provide bond and to retain the tremie con-

crete. Hollow concrete bottom shells, 9 ft high and from 14 to 34 ft in diameter are positioned and partially filled with concrete to bond the structure to the piles. A pair of twin truncated cones, connected by a concrete diaphragm shell, and weighing up to 105 tons is set on top of the bottom shells. Next comes concrete shafts weighing up to 101 tons each that take the pier structure above the water line. The two shafts are connected by a below-water diaphragm. The entire pier assembly is filled with tremie concrete from a floating concrete plant.

Prestressed concrete

Prestressed panels used in concrete roofing job

Contractors and Engineers, V. 51, No. 6, June 1954, pp. 32-34

Describes precast, prestressed panels, made of 6-in. blocks of lightweight concrete tied together with $\frac{5}{8}$-in. and $\frac{1}{2}$-in. steel rods, used to roof a single-story building at Biggs Air Force Base. The use of these factory-assembled panels eliminated the need for shores and under-pinning.

Comments on a prestressed concrete bridge (Bemerkungen zu einer Spannbetonbruecke)

H. Meyer-Larssen, *Der Bauingenieur* (Berlin), V. 28, No. 10, Oct. 1953, pp. 356-358

Reviewed by Aron L. Mirsky

Bridge, replacing a war casualty over the Wuemme at Bremen, consists of a 20 cm thick slab supported by two I-shaped girders 5 m on centers. There are no cross beams or diaphragms. Span is 33.12 m (108.7 ft); widths are 6 m (19.7 ft) between curbs, 9.4 m (30.8 ft) over-all. Design loading is heavy (Class 30). Article is primarily concerned with the design and execution of the prestressing.

Prestressed concrete beams: the economical shape of section

Reginald G. Robertson, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 3, No. 1, Apr. 1954, pp. 242-247

Reviewed by Aron L. Mirsky

Author presents a design chart relating nondimensional coefficients for moment due to superimposed load alone to cross-sectional area of I-shaped beams, in terms of span and allowable concrete stress, for various ratios of web thickness to beam depth. Use of chart, illustrated by an example, results in rapid evaluation of cross-sectional areas and working-load moments, hence rapid preparation of several designs from which the most economical (in terms of costs of cables and concrete) may be selected.

Strain gage tests on prestressed concrete beams

A. T. Waidelich, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. B-1 B-34

Describes briefly tests on full-sized prestressed concrete beams, one 40-ft beam and one 60-ft beam. Also describes use of prestressed beams in a small industrial building.

Prestressed girder in a new bank building (Rahmenbinder aus Spannbeton in einem Bankneubau)

B. Neunert, *Beton- u. Stahlbetonbau* (Berlin), V. 47, No. 11, Nov. 1952, pp. 254-256

Reviewed by Rudolph Fischl

The new building of the Berlin Bank is an eight-story reinforced concrete frame with three bays of about 20-ft span. To provide a hall in the first floor without interior supports, the column load of the upper seven stories had to be supported by a girder of 60-ft span with an available height of only 63 in. With concentrated loads of 800,000 lb at the third points, a prestressed concrete girder (Dywidag system) was chosen as the most economical solution. The girder is part of a fixed frame with a tie at the first floor level and foundations under the basement floor. The cables were prestressed in two operations. First, they were prestressed with half the maximum stress. After the erection of four stories about two months later, the cables were completely stressed and grouted.

Measuring loads in wires. Two simple devices to ensure accuracy in prestressing

N. W. B. Clarke, *Engineering* (London), V. 177, No. 4613, June 25, 1954, pp. 812-815

Reviewed by Aron L. Mirsky

First device (the Cambridge tension indicator) consists of a stiff double-cantilever steel spring with attached dial indicator. Instrument is mounted on a long-line stretching bed with aid of a three-legged reaction stool and two wire grips. Second device (the Building Research Station tensometer) is a dead-weight balance using a bell-crank lever. A tell-tale light is incorporated to indicate to the pump operator when the desired load has been reached; if wire creeps, light goes on again and remains on until desired load has been reestablished by operation of pump. Calibration is against a proving ring. Each device has its comparative advantages and disadvantages which are fully discussed, together with design and methods of use.

Introduction to prestressed concrete theory

HAROLD FEALDMAN, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. C-1–C-29

Presents a concise review of fundamentals and basic methods of calculation involved in the analysis of statically determinate structures in prestressed concrete.

Plymouth's new prestressed concrete footbridge

Surveyor (London), V. 112, No. 3290, July 4, 1953, pp. 457-459

Reviewed by M. W. JACKSON

Describes construction of a 362 ft long bridge at Whitleigh, Plymouth, which consists of a series of simply supported and cantilevered spans supported, in turn, on inclined inverted V-supports. Two suspended spans from the cantilever arms are 70 ft each. The deck is 56 ft above the stream. Main beams, on 11-ft centers, are over 4 ft high; they are made from a series of precast sections, each 6 ft long. The floor consists of precast T-beams supported on the lower chord and prestressed laterally to stiffen the bridge.

The bridge required three tons of mild steel and 4.25 tons of high tensile steel. A bridge of the same spans in reinforced concrete would have required about 50 tons of mild steel.

Theory and design of axially loaded prestressed concrete tension members

FREDERICK WILLIAM GIFFORD, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 2, No. 3, Dec. 1953, pp. 604-608

Reviewed by ARON L. MIRSKY

Author outlines analysis based on exact theory, with effects of creep and shrinkage included, and shows that it is too cumbersome to use in determining the concrete section, but is suitable for checking a section once one is selected. Two approximate methods for obtaining a trial section are indicated.

Prestressing applications expand abroad

Engineering News-Record, V. 152, No. 9, Mar. 4, 1954, pp. 25-26

Reviewed by S. J. CHAMBERLIN

A thin, multi-arch dam planned for Algiers will be prestressed with flat jacks reacting against rock at the downstream toe of the buttresses to maintain the pressure line inside the middle third of the cross section. Prestressed cables will counteract secondary stresses. Flat jacks will also be used for longitudinal prestressing of a runway in the same country. Cables will apply the transverse prestress. Flat jacks have been used for longitudinal prestressing of a 24-ft cylindrical aqueduct in France. Several continuous-girder bridges have been built in Europe recently with either each of the main beams precast as a single unit and later tied together with transverse prestress, or each of the beams fabricated by assembling short units and holding them together with longitudinal prestress. Thin-shell concrete roofs have been prestressed to advantage. Precasting and prestressing have been used together in building construction.

Prestressed concrete beams with high tensile stresses

Engineering (London), V. 177, No. 4592, Jan. 29, 1954, p. 152

Reviewed by ARON L. MIRSKY

Brief report of a warehouse at Bury St. Edmunds, with two interesting structural features. (1) The prestressed concrete roof beams were designed for tension of 659 psi in the concrete under the worst conditions. (2) No bolts or other means of connection are used at the bearing points of the beams, their reactions due to dead load being relied on to hold them in place. Report makes no mention of any provision for possible lateral loading.

Wire-winding machine prestresses beams

Engineering News-Record, V. 152, No. 28, Feb. 25, 1954, p. 41

Reviewed by S. J. CHAMBERLIN

Precast concrete beams 50 ft long for a building in Florida were prestressed by winding steel wires around them at 140,000 psi tension. The wire was looped 92 times around the beam by a method similar to that used for prestressing circular concrete tanks. A coat of lightweight mortar was placed pneumatically over the wires. After erection, and with precast concrete joists set on top, the floor slab and top flanges for the beams were cast simultaneously.

Prestressed shell roofs (Voorgespannen betondaken)

B. W. VAN DER VLUGT and J. J. B. J. J. BOUVY, *Cement* (Amsterdam), No. 15-16, 1954, pp. 247-253

Reviewed by JOHN W. T. VAN ERP

In shell roofs of great span, where the dead load predominates, prestressing is of great advantage. One advantage is that the entire cross section of the thin shell is permanently

under compression, which also eliminates possible future shrinkage stresses, a troublesome aspect of shell construction. Shell construction is exceedingly well fitted to make use of prestressing and it is therefore, surprising that thin shell construction and prestressing have developed independently instead of prestressing having been developed as an outcome of shell construction.

Thickness of cables determines the minimum thickness which can be attained because of coverage requirements. Examples of many different shell roof structures are shown, in barrel-type as well as sawtooth-type roofs.

Prestressed concrete buildings

PHILIPPE A. BENS, *The Engineering Journal* (Montreal), V. 36, No. 12, Dec. 1953, pp. 1613-1622

Reviewed by ARON L. MIRSKY

Descriptions of construction methods used on four prestressed concrete buildings recently constructed in Canada—three large warehouses and an arena. That prestressed concrete is now competitive, and selected on its structural or economic merits, is emphasized.

Properties of concrete

Pulse velocity testing of concrete

W. E. PARKER, *Proceedings*, ASTM, V. 53, 1953, pp. 1033-1043

A review of the use of the Soniscope by the Hydro-Electric Power Commission of Ontario, showing the fields in which it is finding its greatest usefulness and indicating the limitations that have been encountered. Applications described include surveys of cracks in massive hydraulic structures, durability or condition surveys of old structures, study of early strength development in concrete, and laboratory studies involving its elastic and mechanical properties.

Effect of detergents on concrete strength

C. J. MACFARLANE, *Civil Engineering and Public Works Review* (London), V. 49, No. 571, Jan. 1954, pp. 59-61

Reviewed by JAMES MICHALOS

In recent years artificial lubricants have been introduced into concrete to minimize the amount of excess water and to increase workability at low water-cement ratios. This paper discusses the effects of a detergent, "Stergene," used for this purpose. Tests were made and on the basis of the findings it is concluded that the benefits of increased workability outweigh the disadvantage of a small loss in strength.

Use of internal-surface-area measurements in research on freezing and thawing of materials

R. L. BLAINE, C. M. HUNT, and L. A. TOMES, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 298-306

AUTHORS' SUMMARY

Measurements of surface by low-temperature nitrogen sorption have been made on specimens of building stones, bricks, and hydrated cement. Measurements have also been made on crushed limestones and cherts with good and bad service records as concrete aggregates. Of the materials examined, those with higher internal surfaces had, on the average, poorer resistance to freezing and thawing than those with lower surface areas.

Orthodox reinforced concrete and its anomalies (in French)

A. COUARD, *Le Genie Civil* (Paris), V. 130, No. 7, Apr. 1, 1953, pp. 133-134

Measure of characteristics of solids and of soils (in French)

A. COUARD, *Le Genie Civil* (Paris), V. 130, No. 12, June 15, 1953, pp. 225-229

Design of beams of reinforced concrete (in French)

A. COUARD, *Le Genie Civil* (Paris), V. 130, No. 14, July 15, 1953, pp. 272-273

Cracking of beams of reinforced concrete (in French)

A. COUARD, *Le Genie Civil* (Paris), V. 130, No. 21, Nov. 1, 1953, pp. 411-413

Shear under tension or under compression (in French)

A. COUARD, *Le Genie Civil* (Paris), V. 130, No. 23, Dec. 1, 1953, pp. 453-454

Reviewed by M. W. JACKSON

The series deals primarily with the cracking of concrete beams and contains many basic ideas and views.

The April article discusses differences between conventional assumptions in fundamental analysis of reinforced concrete and the actual conditions which occur.

The June article discusses stresses at a point in a mass under various load conditions with relation to the plane of rupture. Formulas are derived for establishing the location and direction of cracks and compared with experimental results, including triaxial tests. The formulas are related to a reinforced concrete beam in its most simple form.

The July article considers the two principal design methods of concrete beams, the elastic method based on a triangular stress distribution diagram, and the rupture method

based on a curved or rectangular diagram. The beam is also considered as a flat arch. These are compared with procedure developed in the previous article.

The November article describes the physical development of a crack in a rectangular beam on two simple supports with a concentrated load at midspan, both with and without stirrups. The practical problem of preventing or reducing the formation of cracks is discussed. This may be accomplished by increasing the tensile strength of the concrete or by properly locating the steel. The latter is considered in detail. A better utilization of the factor of safety is suggested.

The December article proposes test procedure to obtain a better idea of the relative intergranular shearing resistance and intergranular bond. If these tests are made on a sufficient scale, they may cause many engineering problems to be reconsidered on the basis of better information.

Theory of volume changes in hardened portland cement paste during freezing

T. C. Powers and R. A. Helmuth, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 285-297
Authors' Summary

New experimental data are presented on the freezing of hardened portland cement pastes with and without entrained air. They are explained in terms of two mechanisms: (1) the generation of hydraulic pressure as water freezes in capillary cavities and (2) the growth of the bodies of ice in the capillary cavities or air voids by diffusion of water from the gel. Air voids limit the hydraulic pressure and shorten the period during which the ice in the cavities can increase. The closer the air voids are to each other the more effective they are in controlling either mechanism.

Would the strength grading of ordinary portland cement be a contribution to structural economy?

Frederick S. Snow, Frederick M. Lea, Leonard J. Murdock, and Edward Burke, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 2, No. 3, Dec. 1953, pp. 448-479 (including discussion)
Reviewed by Anon L. Mirsky

Four introductory notes, by the above named authors, and discussion. There was little argument that British cements, claimed to be the best and cheapest in the world, exhibited great variations in strength and in rate of hardening (several discussers took pains to differentiate between these two qualities), but there was considerable controversy over whether strength grading of cement would lead to economy (via matching the cement to the required strength of the final product) or to confusion.

Mixtures of and contacts between different cements. Note on the incompatibilities (Melanges et contacts entre ciments differents. Note sur les incompatibilities)

M. Duriez, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 2, Mar.-Apr. 1953, pp. 149-161
Reviewed by Anon L. Mirsky

While portland cement is by nature incompatible with many other cements (*e. g.*, aluminous, expansive, sursulfated), it is normally compatible with many others (*e. g.*, natural, slag, pozzolanic). Under certain circumstances, however, mixtures of portland cement and slag or metallurgical cements may be incompatible: for example, in concrete which is in contact with sea water. The difference in reaction rates may be to blame for the disintegration so often experienced by such structures.

Cracking problem in reinforced concrete buildings

E. M. Rensaa, *The Engineering Journal* (Montreal), V. 36, No. 11, Nov. 1953, pp. 1429-1434
Reviewed by Anon L. Mirsky

Theory and practice of prevention of cracks due to shrinkage and temperature forces. Author distinguishes between macro (cracks) and micro (fissures) and demonstrates that while well-bonded reinforcing steel may actually promote the development of cracking, if present in sufficient amount it keeps the cracks small. Three evils are considered: (1) undue emphasis on architectural concrete surfaces (remedy: use a dry mix to reduce shrinkage, plus a surface finish for appearance); (2) control joints (remedy: use widely-spaced contraction joints, plus sufficient reinforcement between joints); and (3) architectural features which are conducive to cracking (remedy: cooperation between architect and structural engineer).

As author notes, much of the (necessarily empirical) knowledge of crack prevention is difficult to reduce to code provisions which can cover all possible cases. A discussion such as this is thus particularly valuable, and is recommended reading for all practicing engineers (architects too).

Flow of moisture in hardened portland cement during freezing

GERALD PICKETT, *Proceedings*, Highway Research Board, V. 32, 1953, pp. 276-284

AUTHOR'S SUMMARY

A mathematical analysis, based upon idealized assumptions, is made of the flow of moisture between the components of hydrated cement paste during freezing or thawing. The components considered are the gel substance, the capillary pores, and the air-entrained spaces.

It is shown that a change in temperature for temperatures below 0 C gives rise to differences in thermodynamic potentials between gel water; capillary water, or ice; and ice frozen in the air spaces. These potentials result in part from pressures produced by volume changes resulting from freezing and, in part, from differences in the vapor pressures of ice and water. Based on these potential differences the analyses show, for example, that during freezing, in which the temperature is being lowered continuously, flow may first be from capillary to gel, then from gel to capillary, and finally again from capillary to gel. During this same time, flow may be continuously from gel to air space.

By taking into account the change in volume resulting from freezing or thawing of ice in the capillary spaces and the changes in shrinking or swelling tendency resulting from changes in amount of gel water, various volume-time-temperature relations are deduced. In particular, it is shown that water movements and volume changes for a specimen depend not only on the present temperature changes but on the past temperature history of the specimen.

Structures

"Flexible" foundation for a turbo-compressor (in Dutch)

C. W. J. GROOTHOFF, *Cement* (Amsterdam), No. 13-14, 1954, pp. 195-197

Reviewed by JOHN W. T. VAN EEP

Experience with many similar structures has led to the adoption of a "flexible" type foundation structure for these high speed (8900 rpm) machines. Conventional heavy foundation blocks of a practically solid mass of concrete often show signs of marked resonance. In a flexible type of foundation structure consisting of a floor slab on relatively slender columns (preferably of different sizes to eliminate another source of resonance), the fundamental frequency of the structure itself usually remains far below the rpm of the machine. Besides, there is ample clearance and operating headroom for various ducts and pipes. Examples of successful "flexible" foundations are given.

Rehabilitating a concrete viaduct

JOSEPH G. CONRATH, *Consulting Engineer*, V. 3, No. 2, Feb. 1954, pp. 31-33

Reviewed by ARON L. MIRSKY

A 400 ft long, curved viaduct in Erie, Pa., so badly deteriorated due to lack of maintenance on top of faulty construction practices that it was condemned, was returned to useful service by the prepacked concrete method.

Queen Elizabeth II Dock

Engineering (London), V. 177, No. 4592, Jan. 29, 1954, pp. 153-155

Reviewed by ARON L. MIRSKY

General description of oil dock at entrance to Manchester Ship Canal, designed for discharging crude oil from 30,000-ton tankers. Slip forms were used in constructing the lock and dock walls.

Alteration and repair of a reinforced concrete warehouse

STEPHEN REVESZ, *Structural Engineer* (London), V. 31, No. 7, July 1953, pp. 179-187

Reviewed by M. W. JACKSON

A four-story reinforced concrete warehouse constructed in 1919 was completely gutted by fire during air raids in 1940. Superficial repairs in 1942 proved to be ineffective by 1950. Underpinning and other new repair work is illustrated. A valuable addition to an engineer's notebook of repair work experience.

Large-scale multistory housing development at Mitchem, Surrey

D. B. FROWD, J. GETTINGS, and C. D. MITCHELL, *The Reinforced Concrete Review* (London), V. 3, No. 2, 1953, pp. 57-78

Reviewed by C. P. SIESS

A general description of the architectural and engineering design features and construction procedures for a 276-unit housing project of reinforced concrete frame construction.

Construction of reinforced concrete ferry-berth pontoons in the Port of Sydney

Basil Robert Monckton, *Proceedings*, Institution of Civil Engineers (London), Part II, V. 3, No. 1, Feb. 1954, pp. 183-187

Reviewed by Aron L. Mirsky

Based on favorable experience with existing pontoons, new units are expected to have a life expectancy of at least 40 years. Article (an abstract of a manuscript in the Institution of Civil Engineers library) summarizes design, construction, and launching.

Four-story garage in Dusseldorf

Engineer (London), V. 197, No. 5119, Mar. 5, 1954, pp. 363-364

Reviewed by Aron L. Mirsky

Brief description of an interesting-appearing structure designed to park around 500 cars (600 under supervised parking). Floors are supported by 15 sets of three reinforced concrete columns; the ramps, on the exterior of the structure, are hung from the cantilevered roof beams. An adjacent 40-room hotel, supported on T-shaped concrete frames, spans the main entrances and exits of the garage and a gasoline station.

Largest lift-slab operation provides barracks averaging $8 per sq ft

Engineering News-Record, V. 152, No. 8, Feb. 25, 1954, p. 25

Reviewed by S. J. Chamberlin

The three-story, 1500-man barracks for the Navy near Norfolk, Va., has a gross floor area of 215,000 sq ft. After the steel columns were spaced 22 x 20 ft, the second floor, third floor, and roof slabs were cast on top of one another on the first floor slab. Individual lift slabs measured 53 x 76 ft and took in 12 columns on which they were lifted by hydraulic jacks.

Tallest reinforced concrete chimney in Czechoslovakia (in Czech)

K. Hruban, *Inzenyrske Stavby*, V. 2, No. 1, Jan. 1954, pp. 2-7

Reviewed by I. M. Viest

Work was recently completed on a reinforced concrete chimney with the top edge 460 ft above the ground. The chimney was founded on a round reinforced concrete well 52.4 ft in diameter and 32.8 ft deep. The maximum inside diameter of the chimney is 31.2 ft and the minimum inside diameter is 20.3 ft. The wall thickness, not including the insulating brick layer, varies from 13.8 to 5.9 in. Layout, design, and construction methods are described.

Construction of two concrete-lined tunnels for the Machkund Hydroelectric Project, South India

George A. Gauld and Roger J. Newport, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 3, No. 2, Mar. 1954, pp. 111-157 (including oral discussion)

Reviewed by Aron L. Mirsky

Interesting description of the construction of the flume and pressure tunnels and the surge shaft. Project was supervised by only a few Europeans, native labor being successfully trained to operate and maintain the modern equipment.

Prefabricated units for bin construction (Silo's in prefab-elementen)

J. J. Smeele, *Cement* (Amsterdam), No. 9-10, 1954, pp. 123-126

Reviewed by John W. T. Van Ee

These 400-ton bins (for bulk cement storage) were built from prefabricated units 6 ft 6 in. long by 1 ft 8 in. high of channel cross section with a 2 in. wall thickness. Each tier of units was tied together by cast-in-place ribs in which the reinforcement protruding through the ends of the units was embedded and interlaced to take the ring stresses in the bin walls. The pneumatic cement conveying system, operating through the porous bottom of the bin, called for a reconsideration of the conventional system of design loads on the wall.

After considerable research, however, the bins were designed for complete silo action over the full height of the walls with a 15 percent increased load on the lower part. The hydrostatic behavior of the cement in the bottom of the bin, when under pneumatic pressure, causes this increased pressure load.

Flotation method of constructing concrete drainage controls in Malayan coastal swamps

J. E. Bunyan, *Surveyor* (London), V. 112, No. 3225, Dec. 26, 1953, pp. 903-906

Reviewed by M. W. Jackson

A 500-mile length of fertile coastal swamps form the west coast of Malaya, from Singapore to Siam. Many miles of swamps have been reclaimed in past few years. Concrete drainage control structures are built every few miles along the coast to serve as outlets of an inland drainage net, and to prevent entrance of sea water. Construction problems are difficult because the control inverts are several feet below sea level, almost invariably in soft blue mud. Construction

of one typical control is described in detail; it is 123 ft long, 27 ft wide, and the top of the superstructure, is 40 ft above the foundation slab. The general method is to build the first portion in a convenient location, float it into final position and sink it, followed by completion of the superstructure.

Cement factory at Bamburi, Kenya

Engineer (London), V. 197, No. 5123, Apr. 2, 1954, pp. 508-509

Reviewed by Aron L. Mirsky

Plant, with capacity of 100,000 tons per year, will supply Kenya, Uganda, and Tanganyika. All construction was of reinforced concrete by native and Asian labor. Gantry crane hall has arched roof of 86-ft span and 15-ft rise, the arch slab being only 2 in. thick; construction was by traveling forms.

Tiko wharf, British Cameroons

Engineer (London), V. 197, No. 5112, Jan. 15, 1954, p. 99

Reviewed by Aron L. Mirsky

To replace a wharf built in 1911 on cast-iron screw piles, two new wharves were built, founded on "Screwcrete" piles. These consist of corrugated steel casings provided with cast steel screws with reinforced concrete noses; the casings are concreted after driving. The wharf was completed by placing precast beams and deck slabs with cast-in-place concrete connections and pile caps. The main wharf is 448 ft long and 50 ft wide; the lighter wharf 398 ft long and 30 ft wide.

Flat slab floors in a textile storage warehouse (Paddestoelvloeren voor tetielmagazynen te goor)

H. H. Toeter, *Cement* (Amsterdam), No. 11-12, pp. 177-184

Reviewed by John W. T. Van Erp

Floors to carry 300 psf live load were found more economical in flat slab construction than in slab and girder construction mainly because of the reduction in reinforcing steel and formwork costs. Column spacing was 19.6 x 17 ft. Alternate designs were made by moment distribution and by moment coefficients (ACI 318-51). Contrary to popular belief that moment distribution will always lead to the most economic structure, the latter design was found to be the most economic. Floor thickness was 8 in. The roof structure over the warehouse area is of the barrel type, spanning 53 ft. The 4½-in. roof slab is of the so-called fusee type where hollow ceramic tubes are incorporated in the concrete. Considerable weight reduction and improved insulating characteristics are obtained, maintaining structural strength and fireproof requirements.

General

"Elastic" track used in France

Railway Age, V. 136, No. 6, Feb. 8, 1954, pp. 72-73

Reviewed by Aron L. Mirsky

Welded rails bearing on grooved rubber tie pads placed directly on timber or steel-concrete ties and held in place by elastic fastenings (rail clips, griffons) are designed for 23-ton axle loads at speeds exceeding 100 mph. The concrete ties consist of two reinforced concrete blocks (one under each rail) connected by a solid bar of rail steel. Advantage claimed is the reduced transmission of differential movement and vibration of rails.

Fabrication and utilization of pipe of plain and reinforced concrete (in French)

A. Guerrin and G. Daniel, Editions Eyrolles, Paris, 1953, 130 pp., 1200 Fr

Reviewed by M. W. Jackson

This is a companion volume to *Calculation of Pipes of Plain and Reinforced Concrete* (in French) by the same authors (see "Current Reviews," ACI Journal, May 1953, *Proc.* V. 49, p. 880).

In the design of concrete mixes, the material, aggregate, quality of concrete, cement, cement content, and water-cement ratio are covered. In placing concrete in the pipe, the various methods described include the original method of Monier, use of compression, vibration, centrifugal spun pipes, and casting of pipes in place. The text on pipe reinforcing covers advantages of elliptical against circular, double reinforcing, and machinery for fabricating and placing the steel.

Organization of the factory is discussed, with diagrams of a typical layout.

The choice of pipe—circular, ovoid, etc.—the thickness, reinforcement, weight, types of joints, types of support, allowable loads, and factory of safety receive attention.

Field and laboratory methods of testing pipes, with a few typical results, are illustrated with several good diagrams. Methods of placing pipe are described.

The cause of deterioration of pipe is dis-

cussed—mechanical, physical, chemical, electrolytic—in relation to the conservation and maintenance of pipe. A bibliography of 32 items, all French publications, is included.

This book and the companion volume are excellent textbooks on concrete pipe. They cover practically all phases of the subject. The criticism may perhaps be offered that there is not enough detail of any one specific item, but many practicing engineers and contractors say the same thing about all textbooks. The books are no substitute for job experience, but the title, after all, is not "How to build your own pipes and save money."

Economy in the use of structural materials

G. A. Gardner, R. Morton, and L. R. Creasy, *The Structural Engineer* (London), V. 31, No. 9, Sept. 1953, pp. 233-252

Reviewed by C. P. Siess

Symposium inspired by British government encouragement of steel economy. Paper by Creasy on "Comparative Forms of Constructions" discusses relative steel contents and unit costs of various types of structures with both steel and reinforced concrete frames. The advantages of prestressed concrete in maritime structures are mentioned in the paper by Morton.

Cold weather concreting (Beton Herstellung Bei Kuehler Witterung und Frost)

H. Peters, *Betonstein-Zeitung* (Wiesbaden), V. 19, No. 11, Nov. 1953, p. 437

Reviewed by Werner H. Gumpertz

In discussing means of protecting concrete from freezing, the author suggests protection from frost until the concrete is fully dried or until the still damp concrete has attained sufficient strength to withstand the stresses set up by freezing of the free water. As a guide, it is claimed that a fairly wet concrete of 2100 psi strength can withstand about 25 freezing and thawing cycles. The well-known means of protection are listed, and the usefulness of calcium chloride is all but discounted. Construction joints must be treated carefully to avoid major cracking, and overheating of setting concrete may result in thermal cracking later. Author points out that a concrete frozen during the setting process has a better chance of being rehabilitated by thawing than a freshly-set moist concrete. In the latter case, the expansion of the free water during freezing usually causes extensive damage.

Standard specifications for the vibration of concrete (Richtlijnen voor het trillen van beton)

Cement (Amsterdam), No. 3-4, 1953, pp. B1-B7

Reviewed by John W. T. Van Erp

A set of specifications is proposed to cover the entire field of vibration and mechanical compaction of concrete. In an appendix, a summary is given of the various types of tests, the apparatus used therefore and standards pertaining to these. Discussion and comment is invited before final adoption of these standard specifications.

Concrete making at low temperature (in Dutch)

P. W. Scharroo, *Cement* (Amsterdam), No. 11-12, 1953, pp. 170-171

Reviewed by John W. T. Van Erp

Most important methods and measures are summarized as treated in the latest reprint of A. Kleinlogel's book (in German) on this subject. Some of the many points mentioned are: (1) Although the actual process of hydration is only stopped at temperatures below freezing, it is imperative to realize that the hardening of concrete is considerably slowed at temperatures just above freezing. (2) Stripping of forms and shoring should be scheduled according to the change in temperature. (3) Steel should be fabricated in heated spaces. (4) At construction joints the old concrete should be brought above freezing, as should all formwork and reinforcing steel.

Winter concreting (Betonstobning om vinteren)

P. Nerenst, E. Rastrup, and G. M. Idorn, *Direction* No. 17, 1953, Danish National Institute of Building Research, Copenhagen, Denmark, 108 pp. (abbreviated English translation issued)

The recommended practice comprises three sections plus appendices. In Part 1 factors are discussed which affect the qualities of materials in the freezing of soils, of green concrete, and of concrete during hardening. Laboratory tests have been applied in establishing equations representing the progress of hydration with time and at varying temperature. Factors which affect concrete temperature and hydration when slender members are concreted in cold weather are also discussed.

Part 2 consists of diagrams for determination of initial concrete temperatures required to obtain freezing resistance of concrete under specified conditions. The diagrams have been computed for three cases:

concrete produced with no application of heat to materials, heating of mixing water, and heating of mixing water and aggregates. The diagrams can be used for determining the most suitable combination of cement type, initial concrete temperature, and insulation for winter work. Methods are described for determining during construction whether hydration has proceeded far enough for the concrete to resist freezing. This time-temperature relationship is also used in computing when forms can be stripped.

Construction practices in winter concreting are covered in Part 3, together with methods and equipment used for protecting the water supply, heating of materials, and heating of structures during the curing period. The first appendix presents a method for computing the temperature of the concrete mixture from knowing the temperatures of the materials. Based on experience, and through application of the equations established, the second appendix gives a diagram which shows the types of hollow tile floor slabs (based on depth and weight of concrete topping) that can be concreted under various temperatures.

Effectiveness of vibration of concrete

J. M. Plowman, *Engineer* (London), V. 197, No. 5118, Feb. 26, 1954, pp. 302-303

Reviewed by Aron L. Mirsky

Accelerometers embedded in concrete vibrated on a vibrating table, such as used in plants manufacturing precast units, indicated that acceleration of the mix decreases with increasing distance from the sides and bottom. Thus vibration of a casting with just enough acceleration to compact the sides, top, and bottom would result in a casting with a soft and badly compacted interior, comparable to "case-hardening" in metal elements. Vibrating equipment should therefore be designed to have sufficient acceleration for all parts of the concrete mass to be thoroughly compacted.

Research reported in this paper is apparently part of a much larger program, and reviewer hopes future papers will provide answers to such important and intriguing problems as the maximum size of precast units which can be compacted on a vibrating table without danger of overvibrating surface concrete if acceleration is so selected as to compact interior concrete thoroughly.

Hydraulic jacks expand the versatility of slip-form concreting

Construction Methods and Equipment, V. 35, No. 5, May 1953, pp. 64-76

Reviewed by M. W. Jackson

Sliding-form method of continuous placing of concrete walls with one set of forms is being improved with hydraulic jacks. It is being used now for structures of irregular design, with door and window openings. A large number of jacks can be operated simultaneously through remote control by one man. Various projects are shown and the Concretor system jack is explained.

Mechanical considerations in vibrating concrete (in Spanish)

Antonio Angulo Alvarez, *Revista De Obras Publicas* (Madrid), Aug. 1953

Author's Summary

The first of six parts deals with the types of vibrators including centrifugal, unidirectional, and reciprocating. The second part studies centrifugal force of turning vibrators. It includes mathematical formulas for determining forces in some practical cases; forces depend on dimensions of the eccentric weight, angular speed, and the density of counterpoise material. The third part discusses speed of usual vibrators, their frequency range, and methods of measurement, with special mention to the electronical stroboscopic recorder. Part IV deals with the kinematics of every point of a vibrating unit. Special mention is made of vibrating tables, moved by turning and unidirectional vibrators. More efficiency is obtained with turning than with unidirectional vibrators. In such a case, the trajectories of any vibrating point are ellipses. Also, the case of vibrating tables activated by a reciprocating vibrator is studied.

Chapter V is an original study of vibrators coupled to the same frame. The vibrators have electric induction motors, both run at same speed, and are synchronized in different ways for two vibrators, three vibrators placed at the corners of an equilateral triangle, and for three vibrators in line. Part VI deals with the dynamics of vibration such as establishing theoretical relations between mass, force, acceleration, speed, and other factors. The study of energy consumption is treated specially, and also its vectorial representation.